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HNIQUES FOR REHABILITATING PAVEMENTS WITHOUT OVERLAYS – A SYSTEMS ANALYSIS

Vol. 1. Analysis

DEPARTMENT OF TRANSPORTATION
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Final Report

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U.S.

FOREWORD

These reports present the results of research conducted by the Texas Transportation Institute, Texas A&M University, for the Federal Highway Administration (FHWA), Office of Research, under contract DOT-FH-11-9142. This research study was part of FCP Project 5D, "Structural Rehabilitation of Pavement Systems." There are two volumes: Volume 1 describes the analysis used to determine the feasibility of various innovative techniques for rehabilitating pavements without using thick overlays and Volume 2 contains the appendixes of related information.

The research included the development of a systems decision analysis computer program which used utility theory to simultaneously consider seventeen different decision criteria. From this, a total of nineteen techniques demonstrated the capability of solving certain problems better than currently used techniques for rehabilitation.

Copies of the report are being distributed by memorandum to individual researchers. Additional copies may be obtained from the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

Charles F. Scherffey
Director, Office of Research
Federal Highway Administration

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15. Supplementary Notes

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16. Abstract

The objective of the study was to determine the feasibility of a variety of innovative techniques for rehabilitating pavements without using thick overlays and to develop these techniques to the point where they could be implemented. A total of 39 potential techniques were identified. To determine their feasibility a systems decision analysis computer program was developed which utilizes utility theory to simutaneously consider 17 different decision criteria under four main attributes (Cost, Performance, Energy and Impact). Using this utility decision analysis program, a total of 19 techniques demonstrated the capability of solving certain problems better than currently used techniques. Included in the 19 promising techniques are a) use of rejuvinating agents for flexible pavements, b) horizontally-bored sleeper slab and joint restoration for rigid pavements, c) precast joint assemblies for rigid pavements, d) change the location of lane markings, and e) reworked surface of flexible pavement.

For other specific problems the program indicated presently used techniques are better than any of the potential techniques. And finally, the program revealed that 15 potential techniques did not show any promise at this time. Suggestions for further development, implementation, and research have been made.

The report is presented in two Volumes. Vol. 1 contains the analysis and Vol. 2 contains the appendixes. Vol. 1 is Report No. FHWA RD-78-108 and Vol. 2 is Report No. FHWA RD-78-109.

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PREFACE

This report summarizes the results obtained from a research project conducted for the Structures and Applied Mechanics Division, Office of Research, Federal Highway Administration, under Contract No. DOT-FH-9142.

The work was conducted by a team of professional researchers and staff personnel of the Texas Transportation Institute. William B. Ledbetter and Robert L. Lytton served as co-principal investigators. The report was organized and compiled by William B. Ledbetter. Primary responsibility for preparation of the various chapters and appendixes were:

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App. D	J. P. Mahoney
App. I	N. F. Rhodes

The report is published in two volumes. Volume 1 contains the analysis (chapters 1 through 15) while volume 2 contains the appendixes (A through I).

Valuable supportive direction was given by Donald Ader, who performed most of the computer programming; D. J. Teague, who analyzed data and assisted in computer work, Lynette Kuykendall who keypunched and verified data; and Doris Christensen, Barbara Hodge and Loretta Rother, who typed the rough drafts of the manuscript. Final typing and publication of the report was under the direction of Louis J. Horn, Associate Research Editor for the Institute.

Technical supervision and management was under the direction of Mr. Richard A. McComb of the Federal Highway Administration. Mr. McCómb's reviews and suggestions were instrumental in keeping the project headed toward meeting the objectives.

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1. INTRODUCTION

1.1 The Problem

Highways in the United States are being utilized by more and more vehicles. Our population is growing, the number of vehicles is growing even faster, the percentage of commercial vehicles is growing even more, and the traveling public is becoming increasingly dependent upon the nation's transportation system to move goods and people. The vast network of the nation's highways, from the high level interstate system, through the U.S. highway system, the state primary highway system, and the farm-to-market system has been developed through a process of continual construction over the past half century. The total investment has become staggering, easily into the trillions of dollars. As the highway system ages, its repair, rehabilitation, and restoration become more and more expensive. Today, the maintenance expenditures in the United States average $41,000,000 per state (see Appendix - Vol 2), which means the total expenditures in the United States exceed two billion dollars annually. (The Federal Highway Administration estimates somewhere around 5 billion dollars is spent on all roadway maintenance in the U.S.) The problem is how to maintain our investment in the optimum manner. Coupled with the deterioration of the highway system as it ages under the effects of traffic, time, and climate, comes the decline in revenues (largely based on gasoline taxes) and the decline in the purchasing power of the dollar through inflation. All of these factors force transportation agencies to examine more carefully than ever before their maintenance procedures and methods to develop optimal ways to maintain the nation's highways. This does not mean that optimal solutions were not always sought, but rather that increasingly effective solutions are <u>always</u> sought - as the process is one of continual improvement as technology advances and priorities change. Maintenance is increasing in importance as we begin to shift from less new construction to more maintenance. So, new and fresh examinations into ways to rehabilitate pavements are needed today - and will be periodically needed in the future to make sure that the best possible decisions are continuing to be made.

In general, there are three different ways a deteriorated pavement may be rehabilitated. They are:

1. Tear out and replace
2. Overlay (perhaps preceded by some improvement)
3. Rehabilitate in place without overlaying

Present practice utilizes the first two methods extensively. The first is simply reconstruction and currently there are research efforts underway to reuse the old pavement materials (recycling). The second is a tried and true method of rehabilitating, but a disturbingly large percentage of the overlays do not work as well as desired. Furthermore they are expensive and energy intensive, which are undesirable.

The present method of making pavement rehabilitation decisions considers two factors: construction cost (first cost) and performance (life and expected maintenance cost). The decision maker considers these two factors in a quantitative way and considers, in a subjective (qualitative) way, other important factors such as traffic inconvenience, energy, safety, pollution, and noise. For example, he may require 24 hour-a-day construction on an urban freeway to reduce lane closure time and he may specify a more expensive overlay in such a situation so as to prolong the time before future rehabilitation is necessary. In a rural situation longer construction times are often permitted. Thus, qualitative-type considerations often are examined in the decision-making process.

But, as stated earlier, transportation agency decision makers can no longer rely on accepted practices and still accomplish the enormous task of maintaining and improving our highways. Given today's conditions, he needs (1) other alternatives to extensive reconstruction or overlay and (2) a better way to consider all the factors involved in any rehabilitation problem.

These two elements are addressed in this report.

1.2 Objective and Scope

The objective of this study was to determine the feasibility of a variety of innovative techniques for rehabilitating pavements without using thick overlays and to develop these techniques to the point where they could be evaluated in the field.

Plausible techniques of rehabilitation with minimal disturbance to the existing pavement structure were evaluated, taking maximum advantage of the inherent strength and remaining life of the existing pavement structure. Recommendations of methods thought to be the most promising for further research and evaluation have been made.

The research under this contract was specifically directed away from the following classes of problems:

1. Reflection cracking
2. Drainage failures and associated rehabilitation techniques
3. Reuse of pavement materials (recycling)

1.3 Distress Mechanisms and Manifestations

In order to meet the objectives of this project, pavement distress mechanisms and manifestations must be identified, defined, understood, and related to the particular techniques considered.

This was a complex task because there are as many different definitions and understandings of pavement distress as there are different transportation agencies. There are no generally accepted standard definitions! So an early task became to select those definitions which would enable us to most closely meet the objectives of the project. To do this, the definitions developed in the HRB Special Report 113 (1)* were compared with those developed in NCHRP Synthesis 9 (2) and the research teams' experience (3 - 7). For the two general types of pavement structure, flexible and rigid, the following distress manifestations and probable mechanisms were selected and defined:

1. Flexible Pavements

1.1 Rutting

Description: Longitudinal depressions that form under traffic in the wheel path and have minimum length of approximately 20 ft. (6 m).

Probable mechanism: Localized and channeled wheel traffic over unstable pavement or foundation; traffic heavier than the design strength of the pavement structure.

1.2 Raveling

Description: The progressive disintegration from the surface downward by the dislodgement of aggregate particles.

Probable mechanism: Uneven distribution of bitumen from the spray bar; insufficient bitumen or binding agent in the surface; traffic action

*References contained in chapter 15.

on a weak surface; asphalt cement which has lost some of its adhesive qualities.

1.3 Flushing (Bleeding)

Description: Free bitumen on the surface of the pavement.

Probable mechanism: Excess bitumen; rich application of bitumen with insufficient blotter.

1.4 Corrugations

Description: Transverse undulations at regular intervals in the surface of the pavement consisting of alternate valleys and crests less than 2 ft. (0.7 m) apart.

Probable mechanism: Traffic action on an unstable pavement or on a pavement over an unstable roadbed; surface course too soft to resist shoving, low stability. The low stability of a hot mix asphalt concrete may be caused by too rich a mix, a mix that has too high a proportion of fine aggregate, or a mix which has too soft an asphalt cement.

1.5 Alligator Cracking

Description: Interconnected or interlaced cracks forming a series of small polygons that resemble an alligator's hide.

Probable mechanism: Generally, weak base or roadbed; weakening of pavement caused by embrittlement over a resilient foundation.

1.6 Longitudinal Cracking

Description: A crack or break approximately parallel to the pavement centerline.

Probable mechanism: Cold or improperly constructed joint between pavement sections; reflection of underlying joint; settlement under traffic or shrinkage/swelling of roadbed; possible shrinkage of surface course or insufficient pavement thickness.

1.7 Transverse Cracking

Description: A crack approximately at right angles to the pavement centerline.

Probable mechanism: Shrinkage of the surface courses of pavement structure (often temperature associated) and possible traffic action; reflection of cracks under the surface course or insufficient pavement thickness.

1.8 Patching

Description: Correction made to pavement defects by maintenance forces including sporadic seal coating.

Probable mechanism: Any of the other eight flexible pavement distress types can create a requirement for patching.

1.9 Failures/Mile (potholes)

Description: Bowl-shape holes of various sizes in the pavement; a section of pavement usually confined to less than 20 ft. (6 m), where the surface has been eroded or is badly cracked and depressed - areas may contain loose pieces of material and create a hazardous driving condition.

Probable mechanism: Localized disintegration or failure of the pavement from traffic over weakened spots. These weakened spots often initially start in the base course and are caused by excessive water infiltration through surface cracks, or from adjacent higher ground.

2. Rigid Pavements

2.1 Pumping

Description: The ejection of mixtures of water, clay, or silt along or through transverse or longitudinal joints, cracks, or pavement edges.

Probable mechanism: Insufficient support from water - saturated material underneath, squeezing the water and fines out through the joints and cracks or from under the slab.

2.2 Failures/Mile

Description: A section of pavement, usually confined to less than 20 ft. (6 m), where the surface is badly cracked, may be faulted and/or contain loose pieces of the concrete slab. Hazardous to driving. Also localized shattering or buckling of the slab occuring usually at a transverse crack or joint.

Probable mechanism: Heavy traffic overstressing the concrete slab; excessive expansion of the slab with insufficient joint width; joints fouled with incompressible material preventing the joint from serving its purpose.

2.3 Surface Deterioration (Scaling, Raveling)

Description: Progressive disintegration and loss of the concrete wearing surface.

Probable mechanism: Eroding of the surface by reaction from deicing materials; repetitive freezing and thawing cycles or weakened surface caused by overfinishing; heavy traffic.

2.4 Spalling

Description: Breakdown or disintegration of slab edges at joints or cracks or directly over reinforcing steel, usually resulting in the removal of sound concrete.

Probable mechanism: Breakdown of pavement joint edges from traffic action and progressive destruction of the surface adjacent to this damage; possibly weakening of this surface caused by overfinishing of the slab at the joint.

2.5 Longitudinal Cracking

Description: A crack or break approximately parallel to the pavement centerline.

Probable mechanism: Lateral contraction; lateral movement and settlement of the roadbed; possibly lateral bending or curling.

2.6 Faulting

Description: Differential vertical displacement of abutting slabs at joints or cracks creating a "step" deformation in the pavement surface.

Probable mechanism: One slab settling or raising more than the adjacent slab; uneven roadbed support under the slabs from such factors as pumping; heaving of one of the slabs. Faulting often occurs at the junction of a rigid and a flexible pavement.

2.7 Crack Spacing (CRCP Only)

Description: Linear distance between transverse cracks. CRCP should contain cracks approximately perpendicular to the centerline of the pavement. If the crack spacing is approximately 5 to 10 ft. (1.5 to 3.0 m), this is the normally expected and preferred spacing. If the crack spacing is larger than 10 ft. (3.0 m), large crack openings can be expected. If the crack spacing is less than 5 ft. (1.5 m), frequent cracks can occur in a short area which can lead to a failure.

Probable mechanism: Crack spacings longer than 10 ft. (3.0 m), may be caused by insufficient longitudinal steel or too rapid a strength gain. Crack spacings which are shorter than 5 ft. (1.5 m) may be caused

by an excessive amount of pavement deflection which can be due to inadequate thickness, inadequate support, or heavier than anticipated traffic loadings.

2.8 Y Cracking (CRCP Only)

Description: Cracks which intersect other cracks rather than extending across the pavement slab.

Probable mechanism: Overloading of the concrete pavement; inadequate roadbed support.

2.9 Transverse Cracking (Jointed Only)

Description: A crack or break approximately at right angles to the pavement centerline.

Probable mechanism: Insufficient contraction joints or weakened plane joints; overloading an upward curled slab having inadequate roadbed support.

1.4 Pavement Serviceability and Performance

The quantification of the extent of distress manifestations must also be accomplished before the feasibility of given rehabilitation techniques can be properly assessed. Such quantification also proved to be difficult. The literature is full of attempts--from the simple to the complex--to quantify the nature and extent of distress. Beginning with the AASHO Road Test, the pavement serviceability concept was developed (8, 9) and subsequently refined to its present state (6).

Today, pavement evaluation is generally considered in two categories or elements (6):

1. Serviceability - performance concept involving studies of functional pavement behavior
2. Mechanistic evaluation of structural adequacy

In the first category, Serviceability Index, (SI), is defined by a rating scale which ranges from 5 to 0 (9). This concept was initially used at the AASHO Road Test. At the Road Test, a panel of raters traveled various roadway segments and evaluated each section in regard to its serviceability as defined above. Each rater would, in effect, ask himself: "How well would I like to drive over roads just like this section all day long?" With this in mind, the 5 to 0 scale is segmented as follows:

SI Range	Description
4.0 - 5.0	Very good
3.0 - 4.0	Good
2.0 - 3.0	Fair
1.0 - 2.0	Poor
0.0 - 1.0	Very Poor

From recent data collected in Texas, the following average values of Serviceability Index were observed with respect to highway type for flexible pavements:

Highway Type	Average Serviceability Index
Interstate (IH)	3.9
United States & State (US & SH)	3.5
Farm to Market (FM)	2.8

The second category - mechanistic behavior - is not so well developed. A mechanistic evaluation of the pavement structure measures items such as structural capacity, distress, roughness, and more, and attempts to relate them to performance.

Today many agencies use an approach whereby a pavement rating score from 0 to 100 is established. The higher the number the "better" the pavement (10). Deduct points, based on the distress manifestations, are subtracted from an initial rating (representing the pavement's initial or best condition--and thus not necessarily 100). Thus:

$$PRS = C - \sum_{i=1}^{n} \sum_{j=1}^{m} a(T_i, S_j, E_{ij}) \quad \text{- - - - - - - - - - - - - - - - (1-1)}$$

PRS = Pavement rating score

Where C = initial rating number

a ( ) = weighting factor, depending upon distress type T_i, severity of distress S_j, and extent of distress E_{ij}.

The rating scale selected for this project was one based on simple additive deduct points for the type, severity, and extent of each distress manifestation. Table 1-1 contains the deduct values for each type of flexible pavement distress. Table 1-2 contains the deduct values for each type of rigid pavement distress. The rating scale

TABLE 1-1. DEDUCT VALUES FOR FLEXIBLE PAVEMENTS

TYPE OF DISTRESS	SEVERITY OF DISTRESS	DEDUCT VALUES		
1.1 Rutting		Percent of Pavement Area Affected		
		(1-15%)	(16-30%)	(>30%)
	Slight (0 to 0.5 in. rut depth)	0	2	5
	Moderate (0.5 to 1.0 in.)	5	7	10
	Severe (greater than 1.0 in.)	10	12	15
1.2 Raveling		Percent of Pavement Area Affected		
		(1-15%)	(16-30%)	(>30%)
	Slight (less than 10% of surface aggregates dislodged)	5	8	10
	Moderate (10 to 50% dislodged)	10	12	15
	Severe (greater than 50% dislodged)	15	18	20
1.3 Flushing		Percent of Pavement Area Affected		
		(1-15%)	(16-30%)	(>30%)
	Slight (coarse aggregate above surface of asphalt)	5	8	10
	Moderate (coarse aggregate and asphalt nearly on same plane)	10	12	15
	Severe (black surface, few aggregate particles visible)	15	18	20
1.4 Corrugations		Percent of Pavement Area Affected		
		(1-15%)	(16-30%)	(>30%)
	Slight (0 to 1/4 in. depth)	5	8	10
	Moderate (1/4 in. to 3/4 in.)	10	12	15
	Severe (greater than 3/4 in.)	15	18	20

Note: 1 in. = 2.54 cm

TABLE 1-1. DEDUCT VALUES FOR FLEXIBLE PAVEMENTS (CONTINUED)

TYPE OF DISTRESS	SEVERITY OF DISTRESS	DEDUCT VALUES		
1.5 Alligator Cracking		Percent of Pavement Area Affected		
		(1-5%)	(6-25%)	(>25%)
	Slight (hairline, less than 1/8 in. no pumping)	5	10	15
	Moderate (limited spalling and/or pumping)	10	15	20
	Severe (spalling and pumping or cracking 1/4 in. and greater)	15	20	25
1.6 Patching		Percent of Pavement Area Affected		
		(1-5%)	(6-15%)	(>16%)
	Good (adequate performance)	0	2	5
	Fair (marginal performance)	5	7	10
	Poor (patch should be replaced)	7	15	20
1.7 Longitudinal Cracking		Lineal Feet per Station - Lane		
		(10-99)	(100-199)	(>200)
Case I: Cracks Sealed	Slight (less than 1/8 in.)	2	5	8
	Moderate (some spalling or pumping or greater than 1/8 in.)	5	8	10
	Severe (spalling and pumping)	8	10	15
Case II: Cracks Partially Sealed	Slight (less than 1/8 in.)	3	7	12
	Moderate (some spalling or pumping or greater than 1/8 in.)	7	12	15
	Severe (spalling and pumping)	12	15	20
Case III: Cracks Not Sealed	Slight (less than 1/8 in.)	5	10	15
	Moderate (some spalling or pumping or greater than 1/8 in.)	10	15	20
	Severe (spalling and pumping)	15	20	25

Note: 1 in. = 2.54 cm

TABLE 1-1. DEDUCT VALUES FOR FLEXIBLE PAVEMENTS (CONTINUED)

TYPE OF DISTRESS	SEVERITY OF DISTRESS	DEDUCT VALUES		
1.8 Transverse Cracking		Number per Station - Lane		
		(1-4)	(5-9)	(>10)
Case I: Cracks Sealed	Slight (less than 1/8 in.)	2	5	8
	Moderate (some spalling or pumping or greater than 1/8 in.)	5	8	10
	Severe (spalling and pumping)	8	10	15
Case II: Cracks Partially Sealed	Slight (less than 1/8 in.)	3	7	10
	Moderate (some spalling or pumping or greater than 1/8 in.)	7	10	15
	Severe (spalling and pumping)	10	15	20
Case III: Cracks Not Sealed	Slight (less than 1/8 in.)	3	7	12
	Moderate (some spalling or pumping or greater than 1/8 in.)	7	12	15
	Severe (spalling and pumping)	12	15	20
1.9 Failures/Mile		Failures/Mile		
		(1-5)	(6-10)	(>10)
		20	30	40

(Note: All Failures are considered to have a "Severe" degree of distress.)

Note:
1 in. = 2.54 cm
1 mi = 1.6 km
1 sta = 30.5 m

TABLE 1-2. DEDUCT VALUES FOR RIGID PAVEMENT

TYPE OF DISTRESS	SEVERITY OF DISTRESS	DEDUCT VALUES		
2.1 Pumping		Variation in Extent Not Applicable		
	Slight (Visible deposits of material at pavement surface)	20		
	Moderate (Same as "slight" but with minor faulting of the pavement slab)	40		
	Severe (Same as "slight" but with major faulting of the pavement slab)	60		
2.2 Failures/Mile		Failures/Mile		
		(1-5)	(6-10)	(>10)
Note: All failures are considered to have a "severe" degree of distress.		20	30	40
2.3 Surface Deterioration		Percent of Pavement Area Affected		
		(1-15%)	(16-50%)	(>50%)
	Slight (Visible disintegration)	5	10	20
	Moderate (Less than 1/2 in. deep, some coarse aggregate exposed)	10	20	30
	Severe (Greater than 1/2 in. deep, most of coarse aggregate exposed)	20	40	60
2.4 Spalling		Percent of Joints or Cracks Affected		
		(1-15%)	(16-50%)	(>50%)
	Slight (Spalls less than 2 in. wide)	5	10	15
	Moderate (Spalls 2 to 4 in. wide)	10	15	20
	Severe (Spalls greater than 4 in. wide)	20	40	60

Note: 1 in. = 2.54 cm
1 mi = 1.6 km

TABLE 1-2. DEDUCT VALUES FOR RIGID PAVEMENT (CONTINUED)

TYPE OF DISTRESS	SEVERITY OF DISTRESS	DEDUCT VALUES		
2.5 Longitudinal Cracking		Lineal Feet Per Station - Lane		
		(0-99)	(100-199)	(>200)
	Slight (Less than 1/8 in. wide, no faulting or loss of aggregate interlock)	5	10	15
	Moderate (Less than 1/4 in. wide, little or no faulting)	10	15	20
	Severe (Loss of aggregate interlock, faulting and/or spalling)	15	20	25
2.6 Faulting		Number Per Station - Lane		
		(<10)	(>10)	
	Moderate (Steps of less than 1/2 in. in height)	5	15	
	Severe (Steps of greater than 1/2 in.)	15	40	
2.7 Crack Spacing (CRCP Only)		Distance between CRCP Cracks		
		(5-10 ft)	(Other: <5 ft or >10 ft)	
	Closed (Hairline or less than 1/16 in. wide)	0	10	
	Open (Greater than 1/16 in. wide that may show loss of aggregate interlock)	15	40	

Note: 1 in. = 2.54 cm
1 mi = 1.6 km

TABLE 1-2. DEDUCT VALUES FOR RIGID PAVEMENT (CONTINUED)

TYPE OF DISTRESS	SEVERITY OF DISTRESS	DEDUCT VALUES		
2.8 Y Cracking		Percent of Cracks which Intersect		
		(<10%)		(>10%)
	Moderate (Cracks intersect but aggregate interlock maintained)	5		15
	Severe (Loose pieces of the pavement slab are apparent)	15		40
2.9 Transverse Cracking (Jointed Only)		Number per panel		
		(1-2)	(3-4)	(> 4)
Case I: If Joint spacing is less than 20 feet.	Slight (Less than 1/8 in. wide, no evidence of faulting or loss of aggregate interlock)	5	10	20
	Moderate (Less than 1/4 in. wide, little or no faulting)	10	20	30
	Severe (Loss of aggregate interlock, may show faulting and/or spalling)	15	30	40
Case II: If Joint spacing is greater than 20 feet.	Slight (Less than 1/8 in. wide, no evidence of faulting or loss of aggregate interlock)	0	5	10
	Moderate (Less than 1/4 in. wide, little or no faulting)	5	10	20
	Severe (Loss of aggregate interlock, may show faulting and/or spalling)	10	15	30

Note:

1 in. = 2.54 cm
1 mi = 1.6 km
1 sta = 30.5 m

allows for the extent and severity of each type of distress to be related in a quantifiable manner for use in evaluating potential rehabilitation techniques. This system was utilized in the study for the evaluation of each potential technique (see Chapters 3 through 12).

To make use of these tables a "feel" for both the extent and severity of each distress should be obtained. The extent of distress is generally quantified by "percent of pavement area affected" or in some related manner. The severity of distress is generally categorized by using the descriptions of "Slight", "Moderate", and "Severe." If a given distress type is not present on a roadway, the deduct points would be zero for that specific distress.

Figure 1-1 provides three generalized curves of how distress deduct points versus time relationships may appear. Time "zero" is either the time at which the pavement is initially constructed or when some type of maintenance activity has been completed. The curve shown for CASE A indicates that distress deduct points are zero at time equal zero and increase linearly with time. The Case B curve shows that distress deduct points may be a positive value at time zero. This indicates that some pavement maintenance activities after completion may reduce the distress but may not completely eliminate it. This curve also shows that distress may increase nonlinearly with time. The CASE C curve shows that time will be some positive value before any pavement distress occurs. This is particularly true for newly constructed pavements.

Since the AASHO Road Test, another surface characteristic has emerged as being very important - skid resistance! The understanding, measurement, and implications of skid resistance on pavement serviceability as seen by the user has been the subject of a massive research effort during the last 15 years. Notable references in this area include NCHRP Report 37 (11) which sets forth the basic criteria for skid resistance, the special report edited by Rose (12) and the recent work by Gallaway, et al. (13) which reports on the factors necessary to minimize hydroplaning. Standard methods of measurements have been rapidly developed and the recent NCHRP synthesis on skid resistance (14) summarizes the state-of-the-art in this important area of highway design and performance.

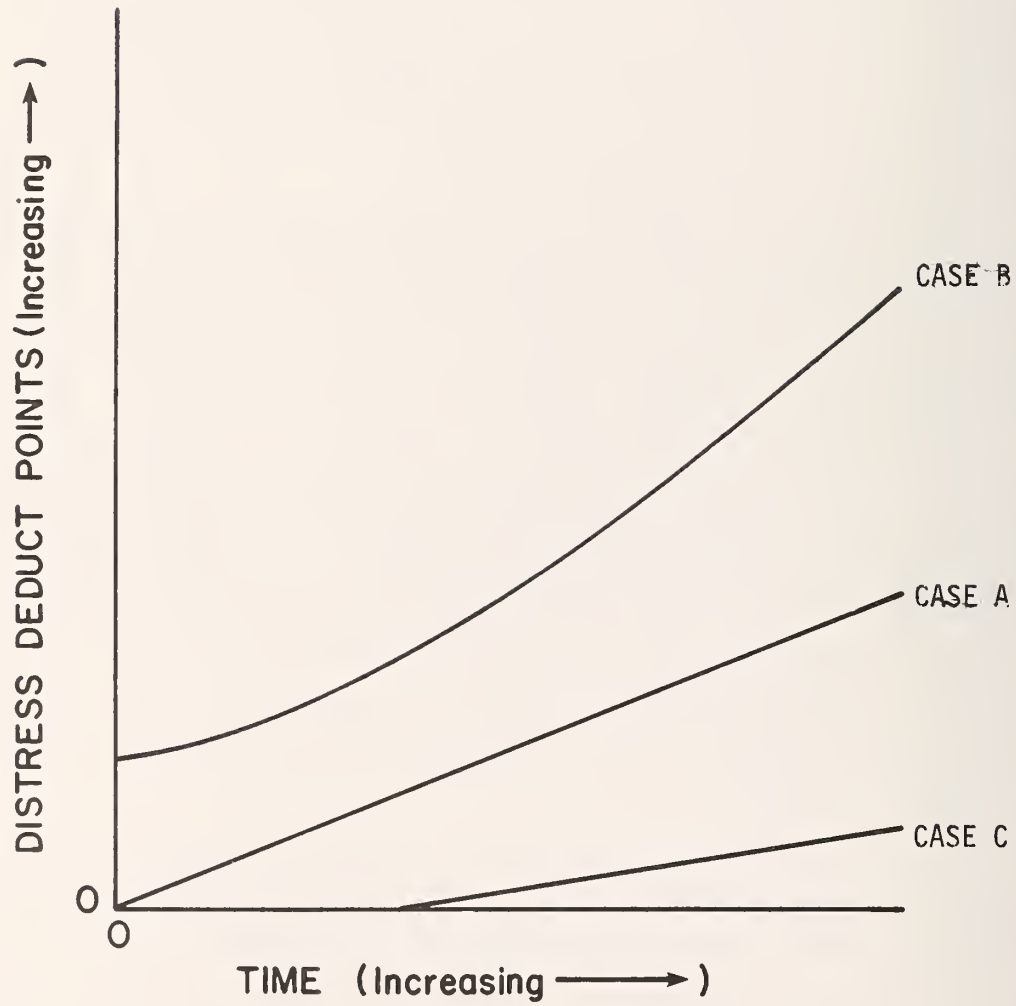


Figure 1.-1. Generalized Distress Deduct Points Versus Time Relationships.

While recognizing skid resistance is a very complex phenomenon, the research team came to the conclusion that certain "accepted" values should be used when comparing various techniques. Thus the following 40 mph (64 km/h) skid number (SN) values were chosen:

	Rigid		Flexible	
	Avg	Range	Avg	Range
Initial	56	40-74	57	40-74
After 5 years	38	23-52	42	22-54
After 10 years	36	18-48	37	15-48

To properly assess the value of potential rehabilitation techniques, certain distress conditions had to be assumed. Depending upon the technique, one or all of the three elements of serviceability and performance (SI, PRS, and SN) had to be determined - both before and after rehabilitation - for each distress type. For comparison purposes, maximum and minimum values for each element were selected. They are given in Table 1-3.

1.5 Research Approach

The research was initiated by performing an intensive literature search in the area of pavement rehabilitation. Computer searches through HRIS and NTIS uncovered over 2000 possible references. After screening, approximately 500 references were found useful. From these references pertinent information relative to many of the techniques were obtained.

Concurrent with the literature search the research team contacted various knowledgeable people from transportation agencies, construction companies, equipment manufacturers, material suppliers, and auxiliary fields to the highway area. Furthermore, an advertisement was placed in the Wall Street Journal soliciting ideas for consideration.

Through the AASHTO Operating Subcommittee on Maintenance the state transportation agencies were queried. In addition to soliciting their ideas, a number of questions concerning their policy, financing, and operation of maintenance were posed. The answers to these questions and an analysis of them is given in Appendix A.

A total of 92 potential rehabilitation techniques were identified. A number were repetitious, and thus combined. Each was given a preliminary

Table 1-3 Maximum and Minimum Values For
Serviceability and Performance and Distress Conditions

Serviceability and Performance Elements	Minimum (worst)	Maximum (best)
1. Serviceability Index (SI)	0	5
2. Pavement Rating Score (PRS)	0	100
3. Skid Number @ 40 mph (SN_{40})	10	80

	Deduct Points	
	Minimum (best)	Maximum (worst)
1. Flexible Pavements		
1.1 Rutting	0	15
1.2 Raveling	0	20
1.3 Flushing	0	20
1.4 Corrugations	0	20
1.5 Alligator Cracking	0	25
1.6 Patching	0	20
1.7 Longitudinal Cracking	0	25
1.8 Transverse Cracking	0	20
1.9 Failures/Mile	0	40

Note: 1 mi = 1.6 km

Table 1-3 Maximum and Minimum Values For
Serviceability and Performance and Distress Conditions (Continued)

	Serviceability and Performance Elements	Deduct Points	
		Minimum (best)	Maximum (worst)
	2. Rigid Pavements		
"All" Rigid	2.1 Pumping	0	60
Pavements	2.2 Failures/Mile	0	40
	2.3 Surface Deterioration	0	60
	2.4 Spalling	0	60
	2.5 Longitudinal Cracking	0	25
	2.6 Faulting	0	40
CRCP Only	2.7 Crack Spacing (CRCP Only)	0	40
	2.8 Y Cracks (CRCP Only)	0	40
Jointed Only	2.9 Transverse Cracking (Jointed Only)	0	40

Note: 1 mi = 1.6 km

screening and analysis by the research team. Those judged to be outside the scope of the project or totally incapable of meeting the stated objectives were discarded. From this screening process a total of 62 potential techniques were considered worthy of detailed evaluation. A complete list of all 92 techniques, together with the reasons for discarding 30 of them, are given in Appendix B.

To standardize the evaluation technique a systems methodology was developed (see Chapter 2). Since construction and maintenance costs had to be estimated for each potential technique, some standard costs had to be selected. To do this several engineering economic assumptions were made. An interest rate of 7% was selected for economic comparisons (see Appendix E). Several indices were selected for updating estimated rehabilitation cost information (Appendix F) and user and accident costs (Appendix G).

To establish typical costs for current maintenance activities, a survey of selected states was made. From these data maintenance costs were determined for most of the currently used techniques (Appendix C). These data were used to compare the value of potential techniques with currently employed techniques.

Finally, because energy considerations are becoming more and more important in every field, rehabilitation decisions should consider the energy implications--both those associated with the rehabilitation strategy itself and those associated with the highway user. A search of the literature revealed a paucity of data compiled in a condensed form for convenient use by rehabilitation decision makers. To overcome this difficulty the scattered energy data were compiled into a convenient form for use on the project (Appendix H).

2. SYSTEMS METHODOLDGY FOR DECISION ANALYSIS

2.1 Utility Methods for Decision Analysis

To determine the feasibility of a variety of innovative techniques for rehabilitation of pavements two approaches might be used.

One approach would be to consider each technique in the light of currently employed judgement criteria; to wit:

1. Is it economically feasible on a first cost basis when compared to existing techniques?

2. Will it cost less to maintain when compared to existing techniques?

These questions involve first cost, expected performance life and maintenance cost - all extremely important in this day of shrinking Transportation Agency budgets and rapidly escalating costs. But these are not the only criteria of importance. In an urban area, the highway engineer intuitively considers the inconvenience of the traveling public in all his decisions. Recently the highway engineer has become more aware of the safety, energy and pollution implications of any decision he makes. The result is that he will not consider any technique which will result in an unsafe pavement (such as one with a low skid resistance), nor one that uses vast quantities of energy (such as the use of cutback asphalts), nor one that pollutes the atmosphere. All of these factors are considered on an intuitive, or "engineering judgement," basis - with generally good results.

The increasing importance of these factors in the decision process concerning the feasibility of any rehabilitation technique suggests a second approach, which is to consider - in a systematic way, all factors affecting the feasibility of any given technique. Early in the course of this investigation an attempt was made to list all the factors which should be considered in the examination of the feasibility of any technique. Seventeen factors were found to be influential to varying degrees. The next logical step was to see if a method could be developed to consider these seventeen factors in arriving at decisions concerning various rehabilitation techniques. After deliberation it was concluded that such a method could be developed, through the application of utility theory, and thus this second approach to meeting the project objective was selected.

Utility theory is neither new, nor is it untried. To the contrary, numerous texts exist (1-4) and the theory has been successfully employed in the analysis of a number of extremely complex engineering problems. For example, the optimum configuration for the supersonic transport was arrived at by use of utility theory (5).

To put it in its simplest form, utility theory is a way to compare apples and oranges with bananas. This involves the following two concepts:

Value - The worth that a person attaches to an object or a service.

Utility - Power to satisfy human wants.

A basic premise in utility theory is that decisions are arrived at using a rational basis, i.e., there is a set of transitive preferences for the outcome of the courses of action available, and the best of the alternatives will be chosen on the basis of maximizing something. Furthermore, these preferences can be rated in terms of their relative power (utility) to satisfy the decision maker's desires.

MacCrimmon (6) has classified the various methods of formally modeling this multiple objective decision making process into: (a) weighting methods, (b) sequential elimination methods, (c) mathematical programming methods, and (d) spatial proximity methods. Sequential elemination methods are ways of eliminating alternatives from further consideration by comparing their attributes in descending order of preference. Housewives typically use this technique for selecting appliances and cooking ware. As an example, they may first eliminate from further consideration all non-Teflon frying pans. This reduces the number of remaining choices. The next attribute considered might be style of frying pan, and so on, until only one candidate is left.

Mathematical programming uses any of a variety of linear, nonlinear, integer, and dynamic programming techniques to select an alternative or set of alternatives which optimizes a mathematically described objective function.

Spatial proximity methods use graphical or mathematical methods of comparing the attributes of an alternative to those of an ideal and selecting the one which most nearly approaches the ideal. MacCrimmon's classification scheme shows that the weighting methods are the most appropriate models for

the decision to choose a particular rehabilitation method. The weighting methods are further subdivided into (a) trade-offs, (b) simple additive weighting, (c) hierarchical additive weighting, and (d) quasi-additive weighting. Of these weighting methods, hierarchical additive weighting and quasi-additive weighting are the most appropriate. The difference between the two is that heirarchical additive weighting assumes that all of the attributes that enter into a decision are independent of each other whereas quasi-additive weighting allows an interaction between the attributes. An example of this kind of weighting was a study of the development of the Mexico City airport facilities by de Neufville and Keeney (7), where they used multiplicative utilities of cost, capacity, access time, safety, displacement, and noise pollution to make the assessment.

Hierarchical additive weighting was selected for this decision-making process because of its relative simplicity. Care had to be exercised in developing the definitions of the decision attributes and in determining utility functions and weights to keep the attributes as independent of each other as possible. As will be seen presently, this attempt to preserve independence was not entirely successful.

2.2 Attributes and Decision Criteria

In hierarchical additive weighting methods, the decision maker assigns values or preferences to high level objectives and then further assigns values or preferences to the attribute to each objective. In the present decision making process, five objectives were identified and the rehabilitation method which best meets them should be selected to:

1. Minimize costs
2. Maximize pavement performance
3. Minimize energy use or save most energy
4. Maximize safety
5. Minimize noise and pollution

These objectives were considered to be the attributes of the decision and were named the Cost, Performance, Energy, and Impact attributes. Objectives 4 and 5 were combined into the single Impact attribute.

Each of these attributes were further subdivided into 17 decision criteria which contribute to satisfying the objective. Detailed definitions of each criterion will be given subsequently.

I. Cost Attribute
   1. Development cost
   2. Capital equipment cost
   3. Construction cost
   4. Maintenance cost
   5. User benefits

II. Pavement Performance Attribute
   6. Time of development
   7. Life
   8. Level of pavement service
   9. Traffic density - construction time relation
   10. Expected reworkability

III. Energy Attribute
   11. User energy savings
   12. Rehabilitation energy
   13. Material energy

IV. Impact Attribute
   14. Safety improvement
   15. Safety during rehabilitation operation
   16. Noise during rehabilitation operation
   17. Pollution

Figure 2-1 illustrates the way in which these seventeen decision criteria enter into making the final decision.

The decision attributes of cost, performance, energy, and impact are aggregations of preferences of several decision criteria, each of which must be defined carefully in order to preserve the assumption that each of these criteria may be considered independently of all others. The following definitions were used in this study.

I. Cost Attribute. The cost decision criteria included in this attribute include all costs of research and development, marketing and purchasing of

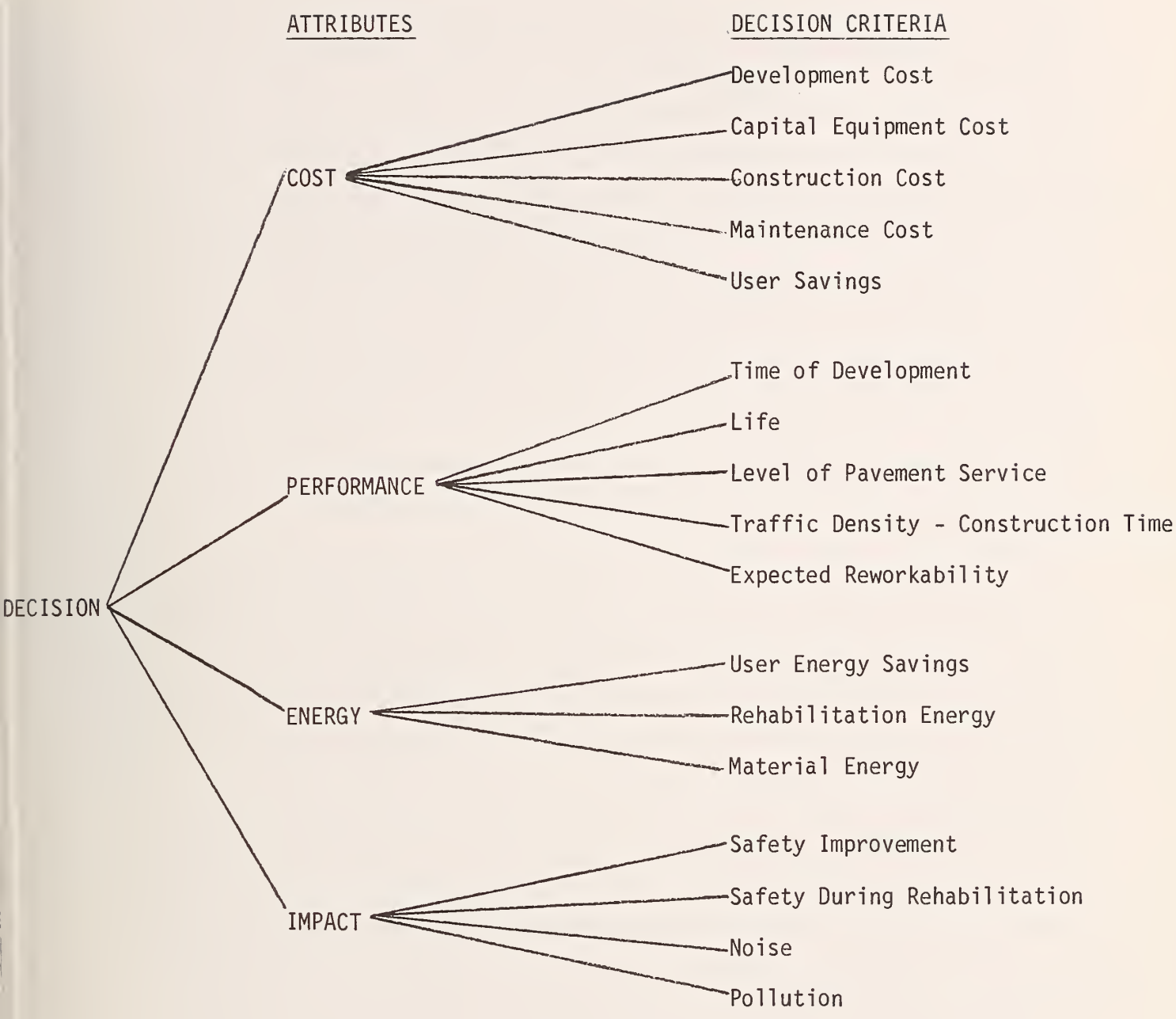


Figure 2-1. Attributes of the Rehabilitation Method Selection Process.

new rehabilitation methods as well as the costs incurred in applying the method to a given pavement, routine maintenance costs subsequent to its application and user benefits which accrue as a result of applying the method. In making a decision on which technique to use, it is not generally correct simply to add all costs to get a total cost because the decision maker will weight several of these costs more heavily than others in making his decision. The weights he applies will depend upon his background and point of view. In this project, it has been assumed that the decision maker is an official of a state transportation agency.

1. Development Cost. This is the total cost incurred from the inception of an idea until the time the technique has been proven applicable. Such costs include: (a) feasibility study; (b) engineering, research and development; (c) legal fees including patents, incorporation, etc; (d) prototype construction and testing; (e) marketing the technique; and (f) tooling up for production.

2. Capital Equipment Cost. This is the total cost incurred in acquiring all "special" equipment necessary for implementing the technique, and all costs associated with marketing the equipment. Such costs include: (a) equipment cost, including purchase, transportation, and special handling; (b) equipment storage, insurance, safety and security; and (c) marketing and management costs.

3. Construction Costs. These are the total costs per lane mile (1.6 km) incurred in applying the technique. Restorative rehabilitation will generally cost more than preventive rehabilitation. Such costs include: (a) material; (b) labor; (c) equipment ownership or rental (excluding special equipment); (d) all equipment operating costs including fuel and maintenance; and (e) overhead, contingencies, and profit.

4. Maintenance Cost. This is the annual cost per lane mile (1.6 km) of maintaining the rehabilitated pavement for the life of a technique after the technique has been applied.

5. User Savings. As defined in this report, user savings (or benefits) are the difference in present value of annual costs per lane mile (1.6 km) incurred by the user between the rehabilitated and non-rehabilitated pave-

ment conditions. Costs encountered if there were no rehabilitation would include (a) vehicle ownership and operating costs, (b) time delay due to roughness, (c) and accident costs. If the rehabilitation technique was used the costs would include (a) time delay around detours during construction, (b) Maneuvering costs and costs of increased accidents during construction (c) vehicle ownership and operating costs (presumably less than if no rehabilitation were performed), (d) reduced time delay due to roughness, and (e) accident costs. User benifits consist of the discounted present worth of accumulated user costs for the life of the rehabilitation method and user costs during construction which are then annualized over the life of the rehabilitation method. The use of time delay due to roughness as part of user costs may be controversial, especially since the reduction in legal speed limit to 55 miles per hour (88.5 km/hr). Studies by Winfrey (8), McFarland (9), and Kher, et al., (10) all indicate a reduction in travel speed with increasing road roughness, but these data were all collected before the reduced speed limit was imposed by law. Unpublished data collected in Texas subsequent to the speed limit change (11) do not indicate a strong dependence of speed on serviceability index. Thus, the assumed relations, which are based upon McFarlands's work, may by somewhat questionable at present and this uncertainty will be reflected in the decision weights assigned to the users' benefits criterion (sec 2.6).

Further uncertainty is associated with the effect of traffic variables on the resulting serviceability index - user cost curve. Undoubtedly, the curve will shift toward higher costs as the volume-to-capacity ratio increases. Such a shift is illustrated in Figure 2-2 for urban and suburban divided highways. Because other factors such as variations in time delay cost can cause similar shifts in the user cost - serviceability index curve, it may be more realistic to recognize that there will probably be a band of such curves, the width of which is specified by a coefficient or variation.

User costs may be estimated using graphs in Oglesby's text book (12) which first appeared in NCHRP Report 133 (13). Costs during construction include: cost of traffic speed changes in the vicinity of the rehabilitation operation, costs of vehicle maneuvering, additional accident costs, and

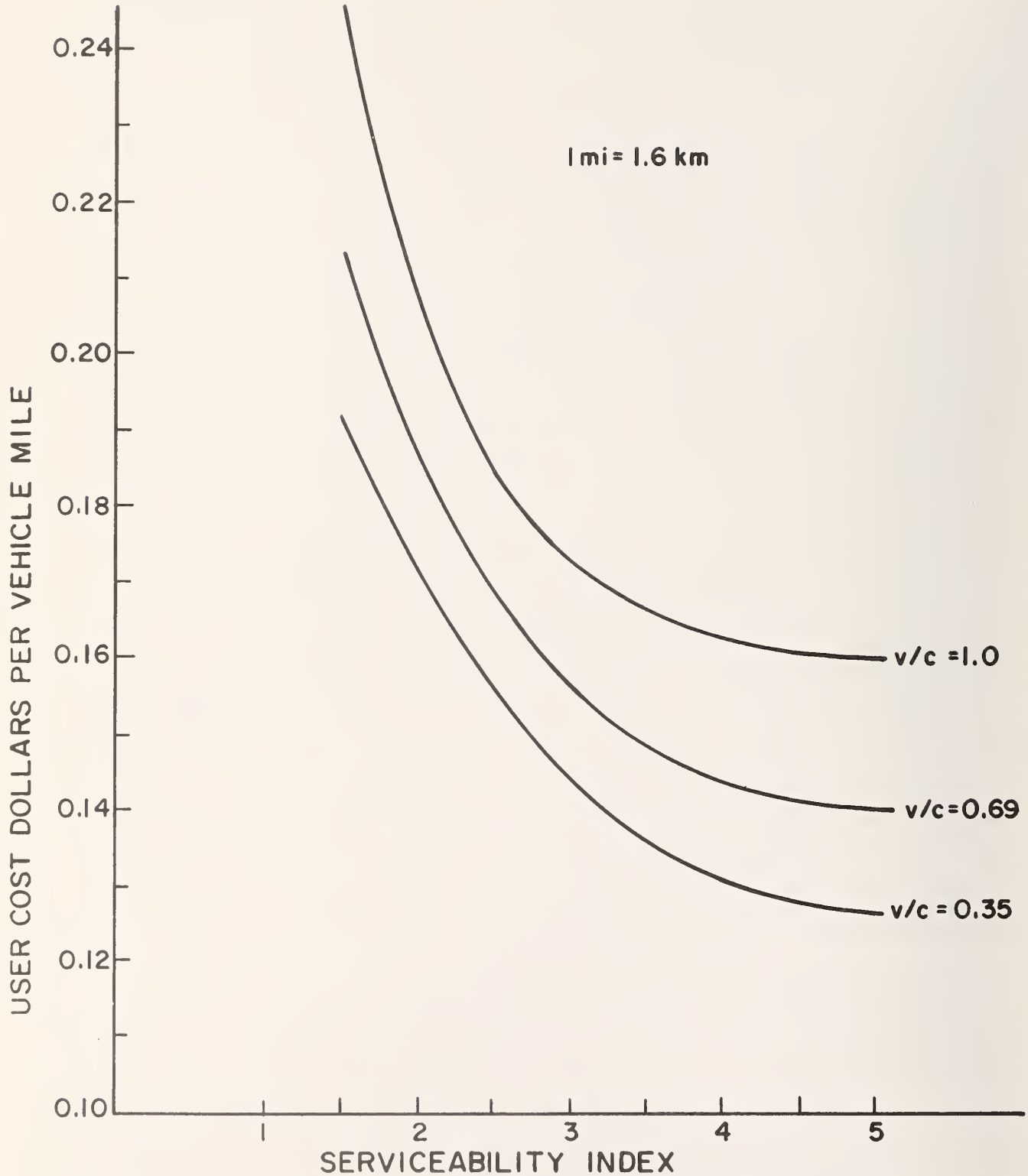


Figure 2-2. Variation in Total User Costs With Serviceability Index for Divided Highways in Urban and Suburban Locations (McFarland-1972 Data).

From Reference 9.

roughness costs of a detour. Costs during the life of the rehabilitation method will be roughness costs which depend upon the expected serviceability history of a pavement. A more detailed explanation of these costs is contained in the Users Guide to the computer program which was written to aggregate the weighted decision attributes (Appendix I).

II. Pavement Performance Attribute. Five separate performance attributes were identified as decision criteria: the time required to develop a new idea to the point where it has been proven applicable, the expected service life of a given rehabilitation method, the level of service that the pavement provides to the traveling public, the amount of time it takes to apply the method and the number of future construction and rehabilitation actions left open as a result of applying a given method.

For the purpose of this report, the life of a technique was considered to be independent of the level of service of the pavement although, in fact, the two are closely related. The decision maker is not only concerned with how long a given method will last but also how well it performs its required function during its life. As an example, it could be supposed that two techniques will last the same time but one will have a very rough riding surface and the other will be smooth riding. The obvious preference is with the smooth pavement. As another example, identical pavements rehabilitated by two different methods will suffer the same amount of cracking during their useful service life but one loses its skid resistance twice as quickly as the other. The pavement with the longer life skid would be preferred.

These examples show how it is possible to consider life and level of service as independent criteria in determining preferences of rehabilitation methods.

Another useful criterion is the traffic volume - construction time relation. A method that is perfectly acceptable when applied on rural highways may be totally unacceptable when applied to an urban highway because it takes such a long time to apply. At first sight, one would be tempted to say that public irritation due to excessive construction delays is adequately measured by user costs and benefits.

However it will be recognized that the amount of time construction

forces are actually engaged in applying a given technique has actually been used as a criterion of acceptability for proposed urban freeway rehabilitation methods without taking user costs into account. For example, 48 hours was the criterion used in a recent NCHRP study of rapid rehabilitation methods (14). Thus, the speed of application, although not independent of user costs in fact, can be considered to be another criterion by which the acceptability of a method can be judged. If two methods generated the same amount of user costs, the one that took the least time to apply would be preferred. The following are the definitions of the five decision criteria considered as part of the "Performance" attribute.

6. Time of Development. This is the time, in years, that will be required to develop an idea to the point where it was been proven applicable.

7. Life. Expected life is the time, in years, that a given rehabilitation technique will keep the pavement at an acceptable level of pavement service or distress which requires no more than periodic routine maintenance. The life of the technique ends when the probability of maintaining an acceptable level of pavement service drops below 50 percent.

8. Level of Pavement Service. This is defined by the critical performance or distress conditions measured over the life of the technique. The number of performance and distress conditions comprising this decision criterion can vary in size depending upon the pavement type and traffic conditions. In order to describe properly each performance condition (serviceability index, skid number, pavement rating score) and distress condition, a maximum attainable and minimum acceptable condition value must be specified. In addition, an "acceptable" life must be selected for each of the two general pavement types (rigid or flexible). The "acceptable" life is the time in years during which management and the traveling public expect the pavement to perform in a satisfactory manner. This is not necessarily the same as the expected life of a given technique, rather it is a desirable pavement life before major revision and/or reconstruction is contemplated. This "acceptable" life is usually determined by experience and is related to the period of time beyond which more funding for additional rehabilitation can be requested for the same section of road without adverse reaction from

management or the legislature. For the purposes of this report acceptable lives are assumed to be in the order of 10 years for flexible pavements and 20 years for rigid pavements.

There are two distress conditions which should be used as input for this decision criterion: the Serviceability Index - Time Relation, and the Skid Number - Time Relation.

9. Traffic Volume - Construction Time. This decision criterion is defined as the ratio of the estimated construction time to the "tolerable" construction time, based on traffic conditions. The tolerable construction time is calculated from equations developed from data in References (12) and (15) and from judgment of the project staff. These equations are graphically portrayed in Figure 2-3. Note the definition of volume/capacity ratio as being 13000 vehicles/lane/day. The tolerable construction time (not to be confused with the estimated construction time) is shown on the abscissa of Figure 2-3 and comes from the traffic levels shown in Figure 2-4. The lower the ratio between estimated construction time and tolerable time the more a given technique will be preferred.

10. Expected Reworkability. This decision criterion describes how easily a pavement can continue to be upgraded, added to, or maintained once the rehabilitation technique has been applied. Obviously, there are some rehabilitation methods (such as post-tensioning existing rigid pavements) that cannot be redone with ease once the rehabilitated pavement has again deteriorated to an unacceptable level of service. This particular method may require a complete reconstruction or a heavy overlay to restore its riding quality. Thus, reworkability of this technique is low and may not be preferred to another method with better upgrading characteristics.

III. Energy Attribute. Three decision criteria are considered here: user energy savings, rehabilitation energy, and material energy. Each of these are defined in detail subsequently.

11. User Energy Savings. In order to determine whether energy is used or saved as a result of the application of a rehabilitation method, the net energy consumption before, during, and after construction must be estimated.

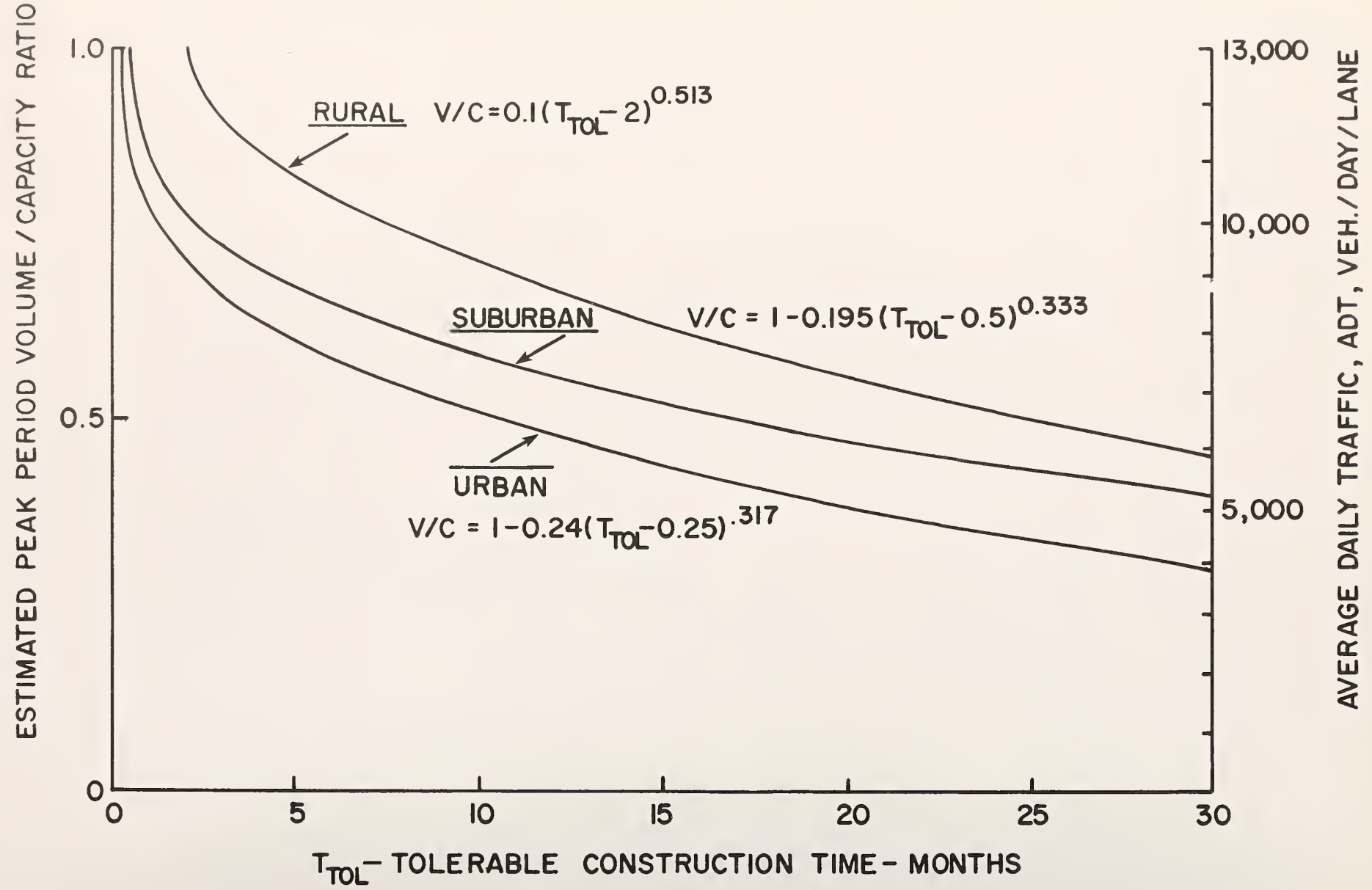


Figure 2-3. Traffic Volume - Construction Time Tolerance.

Figure 2-4. Relation Assumed Between Traffic Level of Service and Tolerable Construction Time (T_{TOL}) in Months.

Level of Service Definition from Ref 16.

Level of Service Illustration from Ref 17.

Level A
Free Flow, no restrictions on maneuvering or operating speed

	Assumed T_{Tol} Months
Rural	⟶ ∞
Suburban	⟶ ∞
Urban	⟶ ∞

Level B
Stable flow, few restrictions

	Assumed T_{TOL} Months
Rural	40
Suburban	30
Urban	15

Level C

Stable flow,

more restrictions

	T_{TOL}, MO
Rural	25
Suburban	10
Urban	4

Level D

Approaching

unstable flow

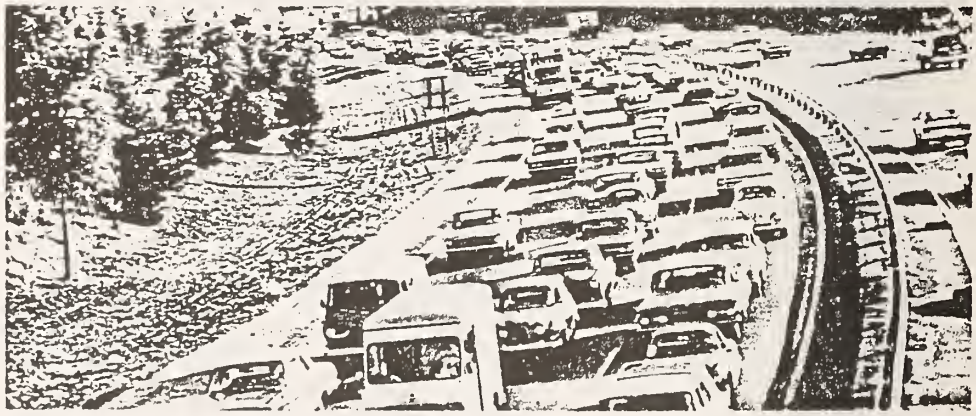
	T_{TOL}, MO
Rural	8
Suburban	5
Urban	3

Level E

Unstable flow,

some stoppages

	T_{TOL}, MO
Rural	3
Suburban	1
Urban	0.5

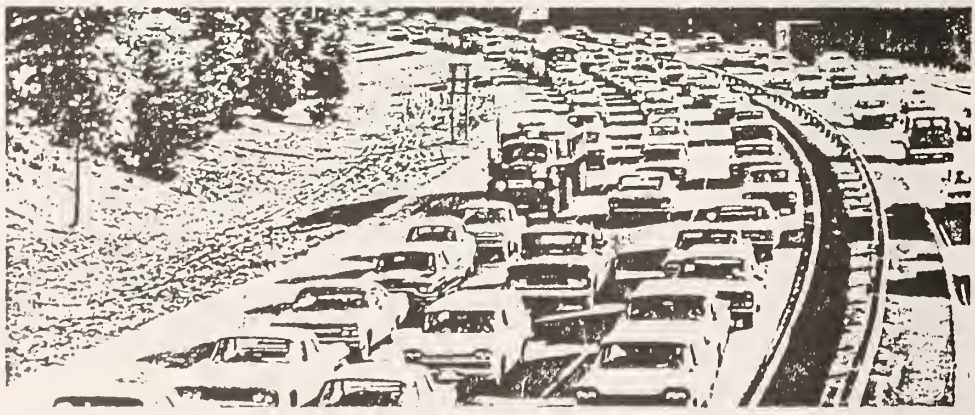


Level F

Forced flow,

many stoppages

	T_{TOL}, MO
Rural	2
Suburban	0.5
Urban	0.25



LEVEL OF SERVICE ASSUMED FOR
PEAK TRAFFIC PERIODS

The energy consumption during rehabilitation activity consists of the fuel equivalents contained in gasoline, tires, and vehicles that are used up in speed changes and maneuvering around the rehabilitation operation. The net energy usage before and after rehabilitation assumes, as shown in Oglesby's text (12), that the cost of fuel usage changes as the road roughness increases. Oglesby states that for smooth roads the roughness factor, F_{DS}, is 1.0. For rough roads, the roughness factor varies with speed as in Equation 2-1 where S = Speed of travel in miles / hr (= 1.6 km / hr).

$$F_{DS} = 1 + 0.01S \qquad (2\text{-}1)$$

For the purposes of this study, it was assumed that F_{DS} is 1.0 when the serviceability index is 5.0 and is 0.002S larger for rough roads when the serviceability index reaches 2.0. The exponential curve that satisfies these two conditions is:

$$F_{DS} = 1.0 + 0.01S \left[1 - 0.676 (SI)^{0.243} \right] \qquad (2\text{-}2)$$

where SI = the present serviceability index

Although the assumed curve is open to some question, it is considered to be a conservative estimate of the roughness factor variation with serviceability index, i.e. the true value of F_{DS} is probably larger at any given level of serviceability index than the value predicted by the exponential equation. Thus, fuel costs and therefore fuel consumption will probably be at least as large for any level of serviceability index as can be calculated using the equation for F_{DS}.

An increase in serviceability index from the old to the rehabilitated pavement over the life of the rehabilitation technique will result in a net savings of energy. After several user energy quantities were calculated using this assumed value of the roughness factor, F_{DS}, it became obvious that the rehabilitation of a pavement usually results in an enormous saving of energy each year compared to what would have been expended if the pavement had not been upgraded. Consequently, it was determined that unless a rehabilitation method saved the energy equivalent of at least 10 gallons, ($3.79 \times 10^{-2} m^3$) of gasoline each year, it was of no practical merit as far as the user energy considerations go. Much better rehabilitation methods should be capable of saving nearly a hundred thousand times as much energy each year.

12. Rehabilitation Energy. This is the energy required to install the technique, and includes such items as transportation, material handling, placing and compaction. It does not include the energy involved in the manufacture of any of the materials utilized. Electrical energy is converted to Btu/yd^2 using the conversion factor of 3412 Btu/kw-hr (1055J/Btu). This energy category was considered essential to distinguish between those rehabilitation methods that use little energy and those that use a large amount. Obviously, a high energy use method will not be preferred by the decision maker unless it has other redeeming features such as safety or speed of operation.

13. Material Energy. This is the energy required to manufacture each material used in the technique. In addition, if the material has an intrinsic energy value (e.g. can be utilized as a fuel) this can be included into the material energy quantity. Whether or not this intrinsic energy is included depends upon the suitability of the material as a fuel source. For example, cut-back additives in asphalt are valuable fuels and thus their intrinsic energy should be included. On the other hand, many asphalts contain chemically combined sulphur which render them unsuitable for use as a fuel (at least until technology advances significantly). In this case, the intrinsic energy value of asphalt should probably not be included. This energy category was included among the decision making factors to recognize the controversy that currently exists between the cement and asphalt industries over the energy-intensiveness of their products. The decision maker will wish to consider carefully and impartially the energy use of each product.

IV. Impact Attribute. This attribute includes four separate decision criteria that are important by themselves but were included as decision criteria under this general heading to preserve the hierarchical ordering used in this decision analysis scheme. The four criteria are safety improvement, safety during rehabilitation, noise, and pollution.

14. Safety Improvement. This is a qualitative measure of the degree of improvement in accident hazard due to the application of a rehabilitation method. A decrease in accident costs would be a good measure of safety improvement if such costs could be predicted as a function of pavement surface properties alone. However, this is known not to be the case since

pavement skid properties are only one of many contributing factors to an increase of accident rates.

15. Safety During Rehabilitation. This is a qualitative measure of level of safety to the traveling public and the work force during the application of the rehabilitation technique. Traffic accident hazards due to chemical, electrial, radioactive, and mechanical energy should be considered.

16. Environmental Impact - Noise. This is a qualitative measure of the expected noise level due to applying the technique. This may be any unusual traffic noise expected because of maneuvering around the rehabilitation activity and any noise level changes due to the rehabilitated pavement surface. The time required to apply the technique will be weighted as well; the longer the rehabilitation construction time, the lower the preference rating of the technique.

17. Environmental Impact - Pollution. This is a qualitative measure of the expected level of pollution due to applying the rehabilitation technique and to any unusual amount of pollution to be expected from re-routing the traffic. Pollution that may result will include solids and gases as well as harmful chemicals that may leach out with time. The time it takes to apply the technique will increase the amount of traffic-related pollution and will tend to lower the preference rating of the technique. There is such a variety of potential noise, pollution, and safety hazards that are possible with any given technique that it was considered necessary to rate them on a descriptive scale that ranged from excellent to hazardous with a probability of occurrence of 0.01 percent. No technique estimated to be more dangerous than this was considered in the assessment.

2.3 Utility Curves

Utility is a measure of preference and as used in this decision making context ranges between 0 and 1, except for the impact criteria of safety, noise, and pollution which are allowed to become negative numbers when a rehabilitation method is judged to be hazardous.

Three characteristic utility curves are shown in Figure 2-5. The curve marked "a" is convex upward and is representative of personal preferences which are willing to take a chance. This risk proneness is due to an

expectation that the method, if successful, will more than justify the expenditure of additional amounts of the independent decision variable plotted along the abscissa. (This abscissa could be any desired unit - dollars, time, serviceability index, etc.) Curve b represents risk aversiveness in personal preferences, indicating a less than optimistic expectation of a successful outcome. Curve c is the risk neutral curve representing a set of personal preferences that is convinced of neither an optimistic nor pessimistic outcome.

Not all utility curves are risk prone or risk aversive for their entire length. In many cases, it is found that up to a certain point, the preference is risk prone and beyond that is risk aversive. This S-shaped curve has a point of inflection at the point where the preference turns from prone to aversive. This type of curve is used more frequently than any other in determining rehabilitation utilities.

The construction of a utility curve normally requires repeated interviews conducted with carefully phrased questions which reveal, by determining levels of indifference, the actual shape of these curves. Since this project was not aimed primarily at determining such decision criteria, alternative methods were adopted which were simpler but still rational. Where frequency distribution data were available, a utility curve was constructed by assuming a normal or log normal distribution and determining a histogram according to the rules in Table 2-1. Where data were not available, project staff were consulted to determine, from their opinions, what the utility curves should be. Some adjustments were made in these curves, once computed data became available, in order that each decision criterion could be made to show a certain amount of distinction between rehabilitation methods. The data for development cost, construction cost and maintenance cost were taken from the results of the survey of State preferences and practices on rehabilitation methods (see Appendix A). The recorded data were given in response to questions like: Would you be willing to pay the following amount for development (or construction) cost? This was followed by a series of cost brackets in which each respondent was asked to check the highest amount he would be willing to pay. The development cost distribution was assumed to be log normal and the construction cost distribution was

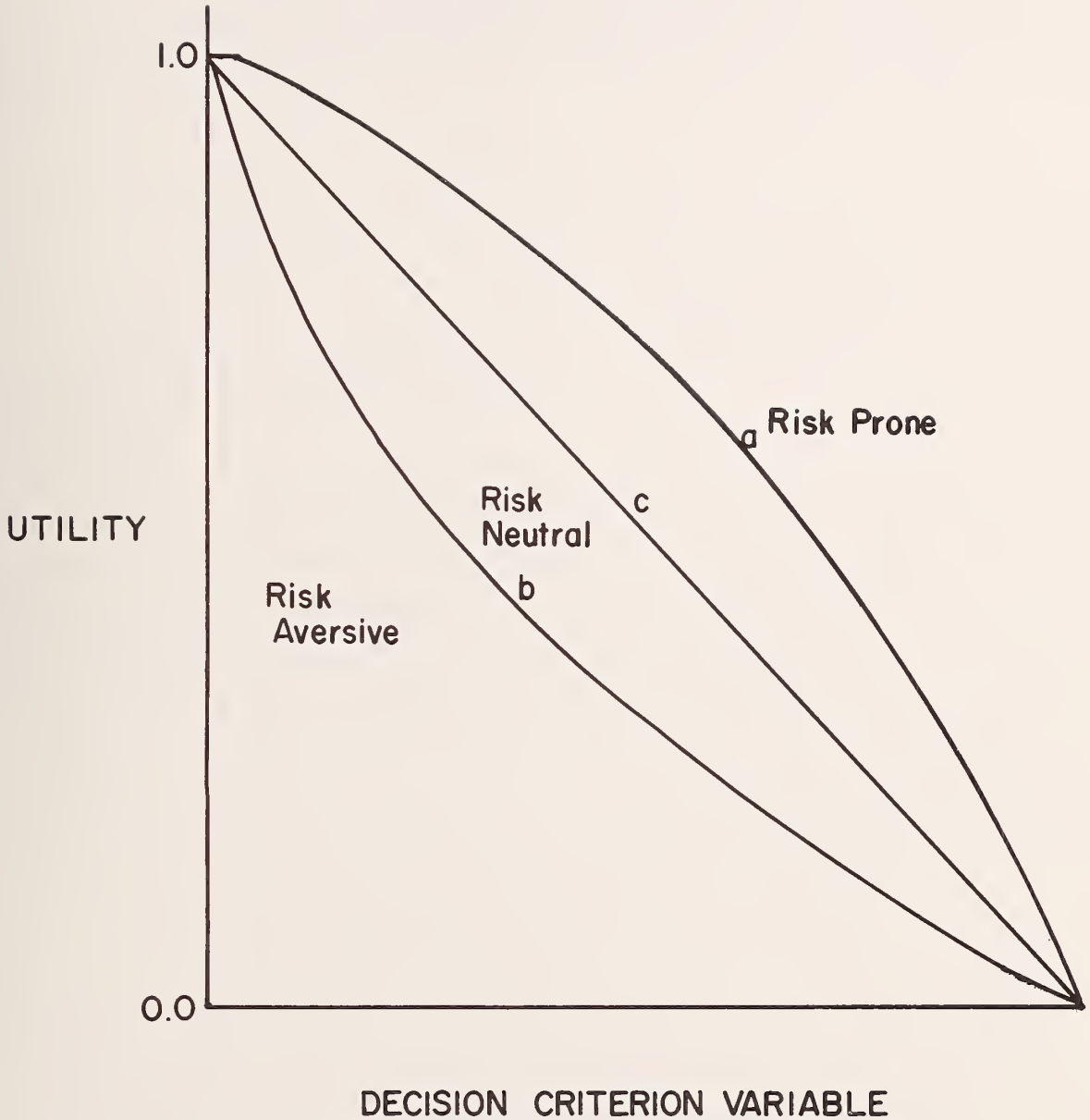


Figure 2-5. Illustration of Typical Utility Curves.

Table 2-1. Rules for Constructing
Utility Curves from Frequency Distribution Data

Value of Decision Variable		Utility Values	
Normal Distribution	Log Normal Distribution	Rising Cumulative Frequency Curve.	Falling Cumulative Frequency Curve.
x_{min}	$\log_{10}(x_{min})$	0.00	1.00
$\bar{x} - \sigma$	$\log_{10}(\bar{x} - \sigma)$	0.16	0.84
$\bar{x}$	$\log_{10}(\bar{x})$	0.50	0.50
$\bar{x} + \sigma$	$\log_{10}(\bar{x} + \sigma)$	0.84	0.16
x_{max}	$\log_{10}(x_{max})$	1.00	0.00

$\bar{x}$ = mean, σ = standard deviation.

assumed to be normal. The cost values and their utilities are given in Table 2-2.

The remaining utility curves were determined by polling the opinions of the project staff and adjusting these in light of computed values as explained previously. Values of the decision criteria variables and the corresponding utilities are given in Table 2-3. Every segment of each utility curve was assumed to be related to its decision variable by an exponential equation of the form:

$$u = a + b\,x^{c}$$

where u = the utility

x = the decision variable

a, b, c, = constraints computed to fit through the points given in Table 2-3.

The S-shaped curves used different sets of constants on each side of the point of inflection.

While the values obtained for these utility curves are considered reasonable no claim is made concerning their absolute validity in all situations. As technology advances and situations are altered, values change. Thus, outcomes may change and different decisions might be made. For the purposes of this project, at this time, the values given were utilized to arrive at decisions concerning the feasibility of each potential technique.

The utility curves determined from the foregoing tables will allow one to determine the utility of any decision variable once a particular value of that variable is known. However, this project considers many rehabilitation methods that are still in the conceptual stage. Furthermore, even if detailed knowledge of a particular technique were known, the actual costs would vary significantly, depending upon the location and extent of the rehabilitation project. It is highly unlikely that exact values of costs, performance, energy, pollution, noise, or safety attributes will be known with much certainty. It is certainly logical to assume that utility values derived from unreliable estimates of decision variables could themselves be unreliable. Futhermore, decisions which are based upon these unreliable utility values could be similarly unreliable.

Table 2-2. Construction, Maintenance and Development Cost Utilities

Decision Variable	Development Cost, $	Log 10 Development Cost,	Maintenance Cost, $/Lane-Mile/YR	Construction Cost, $/Lane-Mile	Utility
x_{min}	10	1.00	250	7,000	1.00
$\bar{x} - \sigma$	1221	3.09	963	9,936	0.84
$\bar{x}$	117,368	5.07	1694	25,132	0.50
$\bar{x} + \sigma$	233,515	5.37	2425	40,328	0.16
x_{max}	500,000	5.70	3500	50,000	0.00
σ	116,147	5.07	731	15,196	

1 mi = 1.6 km

Table 2-3. Decision Variables and Their Utilities

Attribute	Decision Criterion		1	2	3	4	5	Scale on Decision Variable	Utility Characteristic
COST	Development Cost, $	Utility	1.00	0.84	0.50	0.16	0.0	Logarithmic	Falling, S-shape
		Value	10	1221	117,368	233,515	500,000		
	Capital Equipment Cost, $	Utility	1.00	0.60	0.20	0.11	0.0	Logarithmic	Falling, S-shape
		Value	10,000	100,000	300,000	400,000	1,000,000		
	Construction Cost, $/Lane-mi	Utility	1.00	0.84	0.50	0.16	0.00	Linear	Falling, S-shape
		Value	7,000	9936	25,132	40,328	50,000		
	Maintenance Cost, $/lane-mi-yr	Utility	1.00	0.84	0.50	0.16	0.0	Linear	Falling, S-shape
		Value	250	963	1694	2425	3500		
	User Benefits $ / Lane-mi-yr	Utility	0.0	0.75	1.00			Linear	Rising, Risk Prone
		Value	0.0	5000	10,000				
PERFORMANCE	Time of Development, Yr.	Utility	1.0	0.5	0.0			Linear	Falling, Risk Aversive
		Value	0.0	1	3				
	Life , Yr.	Utility	0.0	0.20	0.50	0.88	1.00	Linear	Rising, S-shape
		Value	0.0	3.0	5.0	8.0	10.0		
	Level of Pavement Service	Utility	0.0	0.1	0.25	0.85	1.00	Linear	Rising, S-shape
		Value	0.0	0.4	0.5	0.8	1.00		
	Traffic Density-Construction Time Relation	Utility	1.0	0.9	0.7	0.15	0.00	Linear	Falling, S-shape
		Value	0.5	0.8	1.0	1.5	2.0		
	Reworkability	Utility	1.0	0.88	0.3	0.0	-0.3	Linear	Falling, S-shape
		Value	1.0	2.0	4.0	5.0	7.0		

Table 2-3. Decision Variables and Their Utilities (cont'd)

Attribute	Decision Criteria		1	2	3	4	5	Scale on Decision Variable	Utility Character-istic
ENERGY	User Energy Savings, Btu/yr	Utility	0.0	0.5	1.0			Logarithmic	Rising, Risk Prone
		Value	10^5	10^8	10^{11}				
	Rehabilitation Energy, Btu/s.y.	Utility	1.0	0.3	0.0			Linear	Falling, Risk Aversive
		Value	0.0	50,000	100,000				
	Material Energy, Btu/s.y.	Utility	1.0	0.3	0.0			Linear	Falling, Risk Aversive
		Value	0	5,000	10,000				
IMPACT	Safety Improvement	Utility	1.0	0.88	0.3	0.0	-0.3	Linear	Falling, S-Shape
		Value	1.0	2.0	4.0	5.0	7.0		
	Safety During Construction	Utility	1.0	0.88	0.3	0.0	-0.3	Linear	Falling, S-Shape
		Value	1.0	2.0	4.0	5.0	7.0		
	Noise	Utility	1.0	0.88	0.3	0.0	-0.3	Linear	Falling S-Shape
		Value	1.0	2.0	4.0	5.0	7.0		
	Pollution	Utility	1.0	0.88	0.3	0.0	-0.3	Linear	Falling, S-Shape
		Value	1.0	2.0	4.0	5.0	7.0		

In order to be confident of the decisions made in this condition of uncertainty, the decision variables were described as a distribution of values instead of a single value, as discussed in the next section.

2.4 Prediction of Utility Under Conditions of Uncertainty

Each decision variable was assumed to have a Beta probability density function. At some value of a decision variable x, the probability density is f(x) as shown below.

$$f(x) = \frac{\Gamma(\alpha + \beta)}{\Gamma(\alpha)\,\Gamma(\beta)}\, x^{\alpha-1}\,(1-x)^{\beta-1} \qquad (2\text{-}4)$$

where

$\Gamma(\alpha)$, $\Gamma(\beta)$ = gamma functions of α and β

α, β = distribution parameters which can be determined if the mean and standard deviation of the variable x are known.

The Beta - distribution is very convenient for determining the means and standard deviation if estimates of optimistic, most probable, and pessimistic values of the decision variable are known. The mean is given by :

$$\mu(x) = \frac{1}{6}\,(o + 4m + p) \qquad (2\text{-}5)$$

where o,m,p are the optimistic, most probable, and pessimistic values of x. The variance of a Beta - distribution as given by (α^2).

$$\sigma\chi^2 = \frac{(p-o)^2}{6} \qquad (2\text{-}6)$$

The distribution parameters α and β are:

$$\alpha = \mu\left[\frac{\mu(1-\mu)}{\sigma^2} - 1\right] \qquad (2\text{-}7)$$

$$\beta = (1-\mu)\left[\frac{\mu(1-\mu)}{\sigma^2} - 1\right] \qquad (2\text{-}8)$$

With these values of α and β known, it is possible to calculate f(x) for any value of x.

The expected value of the utility of x is given by the integration:

$$E(\mu_x) = \int_{xmin}^{xmax} f(x)\, u(x)\, dx = \mu_u \qquad (2\text{-}9)$$

and the variance of the utility of x is

$$\text{var } (u_x) = \int_{xmin}^{xmax} f(x)\, u^2(x)\, dx \; - \mu_u^2 \qquad (2\text{-}10)$$

Simpson's rule is used in the computer program to make the two integrations in Eqs. (2-9) and (2-10) numerically. The reason for calculating variances as well as expected values of the utilities is to provide the option of comparing rehabilitation methods at various degrees of certainty. An example of such a comparison is shown in Figure 2-6. Methods A and Method B both have the same expected value of utility but Method B has a broader range of values. Thus, a comparison of the sums of expected value and variance would show Method B to be the preferred method, as would normally be expected. It is even conceivable that a method with a lower expected value but a larger variance would be preferrable.

If the utility distribution is assumed to be normal, comparisons of utilities u at various levels of certainty would require the number z in Eq. 2-11 to be as shown in Table 2-4.

$$U = E(u) + z \sqrt{\text{var } (u)} \qquad (2\text{-}11)$$

where $\sqrt{\text{var } (u)}$ = Standard deviation of u.

As a practical matter, most of the utility variances calculated proved to be so small that comparison of the expected values was considered to be sufficiently representative of overall preferences.

2.5 Combination of Utilities. In accordance with the hierarchical additive weighting method of decision analysis, the individual utilities for each decision criterion must be weighted and added together to give an overall utility for each rehabilitation method. Weighted expected values are added directly according to the following relation:

$$E(U) = \sum_{i=1}^{n} w_i\, E_i(u) \qquad (2\text{-}12)$$

where

$E_i(u)$ = the expected value of utility of the i^{th} decision criterion

W_i = the "normalized" weight of the i^{th} decision criterion

$E(u)$ = the overall expected value of utility

For convenience the weights w_i are "normalized" so that their sum is unity.

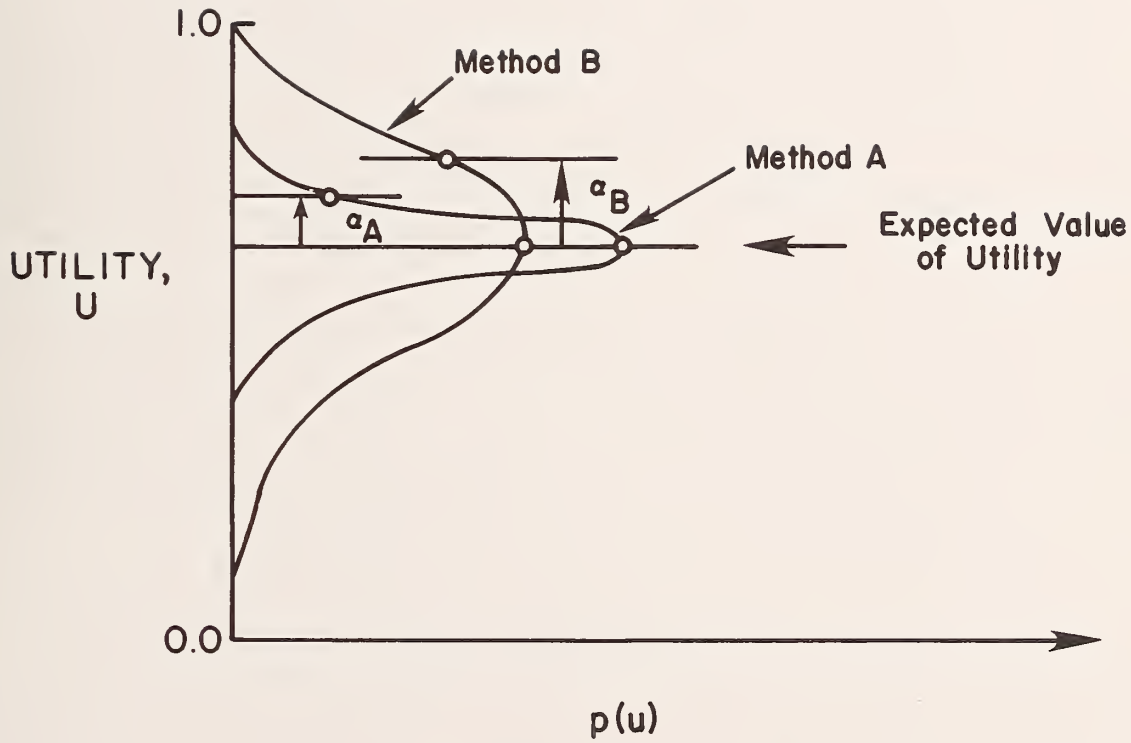


Figure 2-6. Probability Density of Utility.

Table 2-4. Normal Distribution Certainty Factors

Level of Certainty	Z
50	0.00
80	0.85
90	1.28
95	1.64
99	2.33

$$\sum_{i=1}^{n} w_i = 1 \qquad (2\text{-}13)$$

Variances multiplied by the square of their normalized weights all added together to give an overall variance.

$$\text{var}\ (u) = \sum_{i=1}^{n} w_i\ \text{var}_i\ (u) \qquad (2\text{-}14)$$

where $\text{var}_i\ (u)$ = the variance of the utility of the i^{th} decision criterion

$\text{var}\ (u)$ = the overall variance of utility.

By using optimistic, most probable, and pessimistic estimates of the decision variables together with an assumed Beta distribution it is possible to combine in a rational, consistent way the uncertainty that is the usual condition under which decision makers normally make decisions. The utilities thus derived are more reliable estimates of the actual preferences of the decision maker than would be the utilities derived from a set of single estimates of the value of the decision variables.

Assigning consistent weights to the decision criteria requires an in-depth consideration of the point of view of the decision maker. The process of assigning weights used in this project is described in detail in the next section of this report.

2.6 Weighting Factors for Rehabilitation Decisions.

In program UDAREM (Utility Decision Analysis for Rehabilitation and Maintenance), there are three levels of weighting factors which are arranged in descending hierarchical order:

1. Decision attributes
2. Decision criteria
3. Level-of-pavement services criteria

The weights that are applied at each level should reflect the actual preference of the one who is charged with making the decision. In this program, for example, it could be assumed that the decision maker is an official of a state transportation department or a contractor or an equipment manufacturer or a material supplier. Each of these decision makers would weight the various cost criteria differently, based upon their own background and

personal interests. The material supplier and the state transportation official might weight construction costs more heavily in their decision than would the equipment manufacturer and the contractor. As a consequence, the decisions each would make in choosing what they considered the best rehabilitation technique would be different. Because of the subjectivity of the decision making process and the need for consistency in ranking rehabilitation methods, it was decided that the weights attached to each of the three criteria would reflect, as much as possible, the point of view of a state transportation official. Such a person will be concerned with costs, performance, traffic, energy, safety, noise, and pollution just as would any other person but his weighting would reflect a stronger emphasis on the immediate goals of the state transportation agency.

Furthermore his point of view would most certainly be changing with the ever changing conditions under which he operates. For example, since 1973 (the energy crisis) his weighting of energy considerations would be quite different than prior to 1973.

This point of view recognizes that at any given moment a different weight should be applied to each decision criterion depending upon whether the pavement is flexible or rigid, whether a given project is in an urban or rural area, and whether an entire segment of a highway network is being rehabilitated or it is being patched or otherwise repaired in small sections. This suggests that at any given time, there are at least eight different sets of weights that can be applied, one for each combination of pavement type, traffic situation, and extent of rehabilitation.

In decision making terminology, these conditions and subjective influences which affect decisions are termed "states of nature" (2,3). In order to determine weight properly, one must be thoroughly familiar with the definitions of each decision criterion and with the state of nature which influences these decisions. To reduce the complexity of the problem to manageable levels, the time variable was eliminated. Weights have been determined based on present day conditions.

The easiest way to determine weights is to put a 1.0 weight by the

criterion considered to be most important and then establish the remaining weights by asking of each other criterion what relative importance it has in decision making. The relative importance will be expressed as a decimal between 0 and 1. For example, if construction costs have a weight of 1.0 what would the weight of maintenance costs be? Since these maintenance costs will be incurred in the future, they are not usually considered as important as construction costs. A more precise question to be answered might be: Is $110,000.00 of future maintenance costs worth $100,000.00 of construction cost to me at the present time? If the answer is yes, the weight on maintenance costs would be: $\frac{\$100,000}{\$110,000} = 0.91$

In order to achieve some consistency in considering future expenditures, economists calculate the weight of future money on an investment basis assuming an interest rate and a future time when the money would be required in order to calculate a present worth ratio. But in fact, state transportation agencies do not invest this money and let it accrue interest to be available when needed. Instead, they count on future tax revenues and legislature appropriations as they are required with the intent of doing the best they can with the money available. Consequently, future maintenance costs are not, in fact, weighted as heavily in decision making as would be indicated by present worth computations. The same is generally true of user costs, mainly because these costs are not paid directly by the state transportation agency.

The following discussion will consider the weights for each level of decision criteria starting with the lowest order, the level-of-pavement service criterion (number 8 of the 17).

2.6.1 Weights for Level-of-Pavement Service. Each of the eight categories of pavement will be considered in turn. In each of these, one must first identify the important measures of pavement performance and distress and then determine the relative weight of each.

Category 1. Flexible Pavement, Urban Traffic, Segment Rehabilitation. The important measures of performance are considered to be:

1. Serviceability index
2. Skid number

3. Rutting
4. Cracking
5. Pavement rating score

It is considered that the most important of these in the urban situation are skid number; followed by rutting, serviceability index, cracking, and pavement rating score, in this order. The following weights were assigned:

Skid number	1.0
Rutting	0.9
Serviceability index	0.8
Cracking	0.8
Pavement rating score	0.5

The weights were determined by asking the question: "If I have a failing score on rutting on one of these pavements and a failing score skid number on another of these pavements and could do something for only one of them (with money being no object), how much more likely is it that I would choose to correct the skid problem than the rutting problem?"

All other important forms of performance and distress must be compared with the one judged to be most important. In the example shown above, the likelihood of skidding accidents under heavy traffic conditions was considered to be of greatest importance and rutting was second because of the likelihood of causing hydroplaning accidents. Thus, the avoidance of accidents was a primary consideration on urban pavements, while riding quality (measured by serviceability index), pavement leaking and weakening requiring extensive rehabilitation (measured by cracking), and general condition of the pavement (measured by pavement rating score), were ranked in descending order of importance.

By using a comparison between the most important and all others, it was possible to arrive at a weight for each of several measures of distress and performance. These weights were then checked for consistency to see whether there was an equal likelihood of dealing with each of these pavement problems.

In this example, the probable choices of action that would be made with money being no object are shown in Table 2-5.

Table 2-5. Ranking of Pavement Level-of-Service Variables

Process		Probability of Choice(%)	Weight		Probability of Choice(%)	Weight
Ranking	Skid	53 %	1.0	Rutting	47%	0.9
	Skid	56 %	1.0	Ride	44 %	0.8
	Skid	56 %	1.0	Cracking	44 %	0.8
	Skid	67 %	1.0	Pavement Condition	33 %	0.5
Consistency	Rutting	53 %	0.9	Ride	47 %	0.8
	Ride	50 %	0.8	Cracking	50 %	0.8
	Cracking	62 %	0.8	Condition	38 %	0.5

If any of these weights or probabilities seem out of order or too far out of proportion with respect to the others, then the weights could be revised and the level-of-pavement service variables re-ordered.

The same process was followed with each of the eight combinations of pavement type, traffic condition, and extent of rehabilitation.

Category 2. Flexible Pavement, Rural Traffic, Segment Rehabilitation. Under low traffic conditions, future maintenance and rehabilitation costs take on greater importance relative to the possibility of skidding accidents. The decision maker is still concerned with the public welfare but this takes the form of reducing taxes, expenditures, and complaints more than protecting against skidding accidents, although these are still considered important. The important measures of pavement level-of-pavement and their relative weights were considered to be:

1.	Cracking	1.0
2.	Pavement rating score	0.8
3.	Serviceability index	0.7
4.	Rutting	0.6
5.	Skid number	0.5

Category 3. Rigid Pavement, Urban Traffic, Segment Rehabilitation. As in the case of flexible pavement, accident prevention is the primary concern under urban traffic conditions, followed by a desire to preserve pavement condition. Thus the following were assigned:

1.	Skid number	1.0
2.	Cracking	0.9
3.	Spalling	0.9
4.	Surface deterioration	0.9
5.	Pumping	0.8
6.	Serviceability index	0.8
7.	Pavement rating score	0.6

Category 4. Rigid Pavement, Rural Traffic, Segment Rehabilitation.

1.	Cracking	1.0
2.	Spalling	0.85
3.	Pumping	0.85

4.	Surface deterioration	0.8
5.	Skid number	0.8
6.	Pavement rating score	0.7
7.	Serviceability	0.6

Category 5. Flexible Pavement, Urban Traffic, Spot Rehabilitation. Spot rehabilitation on flexible pavement is concerned with specific, localized problems which can be remedied quickly and as inexpensively as possible. It is usually undertaken as a "holding action" with available funds in anticipation of a future major rehabilitation effort. Under urban traffic conditions, the prevention of accidents and loss of vehicle control is considered to be of greatest importance with preventive maintenance as a major secondary consideration.

1.	Skid number	1.0
2.	Flushing (Bleeding)	0.9
3.	Failures / mile	0.9
4.	Alligator cracking	0.8
5.	Raveling	0.7
6.	Corrugations	0.6

Category 6. Flexible Pavement, Rural Traffic, Spot Rehabilitation. Under rural traffic conditions, the aspect of preventive maintenance is still a major consideration.

1.	Failures / mile	1.0
2.	Alligator cracking	1.0
3.	Raveling	0.8
4.	Flushing	0.8
5.	Skid number	0.7
6.	Corrugations	0.6

Category 7. Rigid Pavement, Urban Traffic, Spot Rehabilitation. Where spot maintenance is concerned, the same considerations as mentioned under flexible pavement, urban traffic, apply to rigid pavements.

1.	Skid number	1.0
2.	Failures / mile	0.9
3.	Faulting	0.9
4.	Spalling	0.9

5.	Pumping	0.85
6.	Surface deterioration	0.8

Category 8. Rigid Pavement, Rural Traffic, Spot Rehabilitation. Preventive maintenance and rehabilition becomes relatively more important than accident prevention under rural traffic conditions.

1.	Failures / mile	1.0
2.	Faulting	0.9
3.	Pumping	0.9
4.	Spalling	0.8
5.	Surface deterioration	0.8
6.	Skid number	0.7

The same general process was followed in assessing the weights of the decision criteria which are next in the hierarchical ordering of the decision-making process and in determining the weights of the decision attributes.

2.6.2 Weights for Decision Criteria. There are four sets of decision criteria, each of which comprise the decisive elements of the four independent decision attributes of cost, performance, energy, and "impact." Each of these four sets of criteria will be considered separtely in the following discussion.

Cost Attribute. There are five different kinds of costs which are weighed in deciding what part costs play in adopting a given rehabilitation technique. Since this report adopts the viewpoint of the decision maker in a transportation agency, it will be expected that costs that directly and immediately affect the highway budgeting process will be weighed more heavily than those which can be deferred. These, in turn, will be weighed more heavily than costs which must be incurred by other organizations such as contractors, equipment manufacturers, or material suppliers. The decision maker is neither unaware nor unsympathetic to these requirements for he is certain that eventually a more costly process will be reflected in a rise in prices, which will affect his budgeting process in the future.

The ranking and consistency table for the cost criteria which are considered to reflect a transportation agency's point of view is given in

Table 2-6. Undoubtedly, the tabulated weights could be refined by an appeal to an economic evaluation of present worth of the investments as they affect the transportation agency's budgeting process and this should perhaps be done by transportation agencies in the future to make the decision making process more rational from an economic point of view. However, this degree of detail is widespread in current transportation agency decision making practice. Thus, it was considered logical to retain a certain amount of subjectivity in assessing the weights. As a result of this tabultion, the weights for each cost decision criterion are as listed in Table 2-7.

Performance Attribute. There are five criteria associated with pavement "performance." As strictly defined, performance of a pavement is the history of its present serviceability ratings. However, in searching for a single word that summed up a pavement's intended purpose, the word "performance" seemed most appropriate. (Perhaps a better word should be suggested.)

The decision criteria considered to comprise the performance attribute were adopted as independent measures of how well a given rehabilitation method would perform it's intended purpose. The decision criteria are: the life of a technique once it is applied to a given pavement, level-of-pavement service, time required to develop the technique, the traffic density-construction time relation (which takes into account public irritation at the length of time it takes to apply the rehabilitation technique to a given pavement), and re-workability (which reflects the concern the decision maker has that any future rehabilitation may be much more difficult if a given technique is applied now).

It was considered that public irritation to delays was not always proportional to the amount of money lost in delays, maneuvering, and so on around a maintenance or rehabilitation operation. Users costs could probably be correlated with it but, particularly at higher traffic levels, the user will perceive an eventual benefit and is willing to some extent to suffer the current inconvenience from delays, detouring, queuing, and so on for a length of time that he considers tolerable. Because he can and usually does make this judgment virtually independent of the costs to himself, his ability to

Table 2-6. Ranking and Consistency of Cost Decision Criteria

Process	Cost Criterion	Amount Equivalent to Immediate Budget Worth,$	Weight	Cost Criterion	Amount Equivalent to Immediate Budget Worth,$	Weight
Ranking	Construction	100	1.0	Maintenance	110	0.9
	Construction	100	1.0	Capital Equipment	150	0.66
	Construction	100	1.0	Development	200	0.5
	Construction	100	1.0	User Benefit	300	0.33
Consistency	Maintenance	100	0.9	Capital Equipment	140	0.64
	Capital Equipment	100	0.64	Development	130	0.49
	Development	100	0.49	User Benefit	150	0.33

Table 2-7. Weights for Cost Decision Criteria

Cost Criterion	Weight
Construction	1.0
Maintenance	0.9
Capital Equipment	0.65
Development	0.5
User Benefit	0.33

project into the future and tolerate was considered to be a factor that was virtually independent of user costs.

The question that must be answered in determining the ranking and consistency of the weights to be applied must be asked from the point of view of the transportation agency decision maker. The question to be asked is a bit more abstract than in the previous attribute because what is being considered here is a relative increase in utility, that is, a number ranking between 0 and 1 indicating a degree of satisfaction. In fact, the same kind of determination had to be made in weighting the cost criteria but it was assumed in that case that utility was roughly proportional to cost. In these criteria, a variety of factors were considered which did not have common units of measure. As a consequence, these criteria were compared upon the basis of their increase of utility.

The question to be answered is: "How probable is it that I would prefer an increase of utility of 0.1 utility units of life of a method, as opposed to an increase of 0.1 utility units in time of its development." The resulting table of ranking and consistency is as shown in Tables 2-8 and 2-9 representing urban and rural traffic conditions. As is apparent from the different weights, the traffic situation controlled choices in an urban environment whereas the durability of maintenance and rehabilitation was more important under rural traffic conditions. Table 2-10 gives the final weights determined for both situations.

Energy Attribute. There are three decision criteria included among the energy attributes: user energy saved by applying the method, energy contained in the material, (both in producing or manufacturing it and in its existing value as a fuel), and the energy employed in applying the rehabilitation method to a pavement.

The weights to be applied to these different categories of energy will vary with the point of view of the decision maker. The well-known controversy between concrete and asphalt producers was taken into account here by allowing both the fuel value and the energy used in the manufacture and production of the material used in the rehabilitation method to be added together and weighed in the decision. The transportation agency decision maker

Table 2-8. Ranking and Consistency of Performance Decision Criteria (Urban)

Process	Criterion	Probability of Preference,%	Weight	Criterion	Probability of Preference,%	Weight
Ranking	Life	53	1.0	Tolerable Construction Time	47	0.9
	Life	56	1.0	Level of Pavement Service	44	0.8
	Life	58	1.0	Re-workability	42	0.7
	Life	67	1.0	Development Time	33	0.5
Consistency	Tolerable Construction Time	53	0.9	Level of Pavement Service	47	0.8
	Level of Pavement Service	53	0.8	Re-workability	47	0.7
	Re-workability	58	0.7	Development Time	42	0.5

Table 2-9. Ranking and Consistency of Performance Decision Criteria (Rural)

Process	Criterion	Probability of Preference,%	Weight	Criterion	Probability of Preference,%	Weight
Ranking	Level of Pavement Service	53	1.0	Life	47	0.9
	Level of Pavement Service	56	1.0	Re-workability	44	0.8
	Level of Pavement Service	60	1.0	Tolerable Construction Time	40	0.66
	Level of Pavement Service	66	1.0	Development Time	33	0.5
Consistency	Life	53	0.9	Re-workability	47	0.8
	Re-workability	55	0.8	Tolerable Construction Time	45	0.65
	Tolerable Construction Time	56	0.65	Development Time	44	0.51

Table 2-10. Weights of Performance Decision Criteria

Criterion	Urban	Rural
Life	0.9	0.9
Tolerable Construction Time	1.0	0.66
Level-of-Pavement Service	0.8	1.0
Reworkability	0.7	0.8
Development Time	0.5	0.5

will normally try to maintain a strict neutrality in favoring one or the other material but at the same time is both aware and concerned about the consumption of energy which is becoming more scarce with the passage of time. The weight he attaches to this "material energy" will probably rise in the future.

The same units are used to measure the rehabilitation and material energy (Btu/sq yd or kJ/m^2) but user energy is measured in Btu/yr/mi (kJ/yr/km). Thus, weights must be established, as in the previous attribute, by deciding on the probability of preferring an increase of 0.1 utility units in one energy category or in another.

The ranking and consistency table for energy decision criteria is given in Table 2-11. The weights of energy decision criteria are given in Table 2-12.

Impact Attribute. There are four decision criteria which compose the "impact" attribute: the increase of safety due to the application of the rehabilitation method, the safety to motorists and construction crews during the application of the method associated with the construction of the technique, and the noise and the amount of other pollution involved during and after rehabilitation. Because of the many different factors that must be considered in the areas of safety, noise and pollution, a numerical rating system was used to rate each rehabilitation method. As an example of the numerous possible factors that must be considered, the safety of a technique which is being applied may include hazards to the construction crew from such sources as high voltage, various forms of radiation, heat, high pressure, moving mechanisms, excessive noise, noxious gases, and so on. No other consistent method of considering such a profusion of factors could be found than by the numerical rating. A scale from one to seven was selected using the following subjective definitions.

1. Excellent
2. Good
3. Fair
4. Poor
5. Unacceptable without some precautions ($P < .001\%$)
6. Unacceptable without extensive precautions ($.001\ \% \leq P \leq .01\%$)
7. Totally unacceptable ($P > .01\ \%$)

Table 2-11. Ranking and Consistency for Energy Decision Criteria

Process	Energy Criterion	Probability of Preference,%	Weight	Energy Criterion	Probability of Preference,%	Weight
Ranking	Rehabilitation	70	1.0	Material	30	0.43
	Rehabilitation	80	1.0	User	20	0.25
Consistency	Material	60	0.43	User	40	0.28

Table 2-12. Weights of Energy Decision Criteria

Energy Criterion	Weight
Rehabilitation	1.0
Material	0.43
User	0.27

The use of this scale illustrates one of the desirable features of utility theory. Subjective ratings may be quantified and compared directly with objective values.

As in the previous two attributes, the weights must be found by establishing the probability of preferring an increase of 0.1 utility units of one factor as opposed to another as considered by the transportation agency decision maker. Only in the case of the impact attributes are the utility curves permitted to become negative. These negative values are penalties to allow the decision maker to account for the fact that hazardous techniques may still be used but only with special and perhaps costly precautionary measures taken. Ranking and consistency are given in Table 2-13 and weights of the impact criteria are given in Table 2-14.

The final process in decision making is still one step more abstract since it requires a judgment to be made on the relative weights of each of the attributes, which are themselves combinations of utilities.

2.6.3 Weights on Decision Attributes. When comparing attributes, one determines the probability of preferring an increase of 0.1 utility unit for one as opposed to another attribute. These comparisons are given in Table 2-15.

At this level of abstraction the final decisions must be made on which attributes must be weighted the most heavily. Those persons devoted to energy conservation or environmental protection would probably apply different weights to these attributes than would the transportation agency official, and different weights may result in different decisions. Another extremely valuable result of utility theory is that for the first time the true effects of differing preferences on various attributes can be quantified, examined, and analyzed in a rational and logical manner. For example, the results from a "fervent" environmentalist could be compared with the results of a material supplier. With each giving weights to all the decision criteria they would then know whether or not their decisions would be really so different after all. Interesting possibilities!

The weights determined in this report reflect the point of view of a transportation agency decision maker who will necessarily be most immediately

Table 2-13. Ranking and Consistency of Impact Decision Criteria

Process	Impact Criterion	Probability of Preference,%	Weight	Impact Criterion	Probability of Preference,%	Weight
Ranking	Construction Safety	60	1.0	Traffic Safety Change	40	.66
	Construction Safety	70	1.0	Pollution	30	.43
	Construction Safety	80	1.0	Noise	20	.25
Consistency	Traffic Safety Change	[illegible]	0.66	Pollution	40	0.43
	Pollution	60	0.43	Noise	40	0.29

Table 2-14. Weights of Impact Criteria

Criterion	Weight
Construction Safety	1.00
Traffic Safety Change	0.66
Pollution	0.43
Noise	0.27

Table 2-15. Ranking and Consistency of Cost, Performance, and Energy Attributes

Process	Attribute	Probability of Preference,%	Weight	Attribute	Probability of Preference,%	Weight
Ranking	Cost	0.53	1.0	Performance	0.47	0.9
	Cost	0.60	1.0	Energy	0.40	0.67
	Cost	0.67	1.0	Impact	0.33	0.50
Consistency	Performance	0.57	0.9	Energy	0.43	0.68
	Energy	0.57	0.68	Impact	0.43	0.51

concerned with costs and pavement performance. This does not mean that he is unaware or unconcerned about the other two attributes of his decision, but at present energy and impact do not as immediately affect the budgeting process for which he is primarily responsible. The weights of the decision attributes which have been selected are given in Table 2-16.

2.6.4 Weights Used In This Project. The preceding discussion presented a method for establishing relative weights on decision criteria and attributes from the point of view of a hypothetical transportation agency which was referred to in this report as X-DOT. Project staff reviewed the weights established by this exercise and found their own subjective preferences different from those just presented. We were strongly of the opinion that these preferences would vary depending upon whether the traffic situation was urban, suburban, or rural. Consequently, the entire weighting exercise was repeated for the differing traffic situations to determine weights that were in accordance with the subjective preferences of the project staff. The resulting weights are given in Table 2-17. These latter weights were used to establish the utilities upon which the recommendation of this report are based.

These preferences tend to weight user benefits, level of pavement service, user enerly savings, safety improvement, noise and pollution impact more heavily than X-DOT. On the other hand, rehabilitation energy was weighted less heavily.

The input users guide for this program is given in Appendix I.

2.7 Utility Decision Analysis For Patching

The decision criteria for localized rehabilitation methods required a slight revision in approach because of their localized nature. The major areas where the decision makers' preferences are altered are: construction cost, user benefits, level of pavement service, and user energy savings.

2.7.1 Construction Cost Utility Curve. A new utility curve was constructed for localized rehabilitation construction cost. Two approaches were tried before one was decided upon.

The first approach was to consider the construction cost on a spot basis. Calculations indicated that patching a 6-inch deep hole in a pavement by hand labor may cost around \$20/ yd^2 (\$17 / m^2). An efficient machine-

Table 2-16. Weight of Decision Attributes

Attribute	Weight
Cost	1.0
Performance	0.9
Energy	0.67
Impact	0.5

Table 2-17. UDAREM WEIGHTING FACTORS

	XDOT	URBAN	SUBURBAN	RURAL
COST	1.0	0.8	1.0	1.0
Development Cost	0.5	0.5	0.5	0.5
Capital Equipment Cost	0.65	0.6	0.6	0.6
Construction Cost	1.0	1.0	1.0	1.0
Maintenance Cost	0.9	0.9	0.9	0.8
User Savings	0.33	0.7	0.5	0.4
EXPECTED PERFORMANCE	0.9	1.0	0.9	0.8
Time of Development	0.5	0.5	0.5	0.2
Life	0.9	1.0	1.0	0.9
Level of Pavement Service	0.8	1.0	1.0	1.0
Traffic Volume - Const. Time	1.0	1.0	1.0	0.4
Expected Reworkability	0.7	0.8	0.7	0.6
ENERGY	0.67	0.6	0.7	0.7
User Energy Savings	0.27	1.0	1.0	1.0
Rehabilitation Energy	1.0	0.7	0.8	0.9
Material Energy	0.43	0.5	0.5	0.5
IMPACT	0.5	0.7	0.6	0.5
Safety Improvement	0.86	0.9	0.9	0.9
Safety(During)	1.0	1.0	1.0	1.0
Noise	0.27	0.7	0.6	0.4
Pollution	0.43	0.8	0.7	0.5

laid patch may cost as little as \$4 / yd^2 (\$3.30 / m^2). Utility values of 0.5 and 1.0 were assigned to these two costs and the remainder of the utility curve was estimated. Beyond the \$20 / yd^2 figure, the utility curve was constructed to be risk aversive.

The second approach was to consider patching as a maintenance operation that would be applied to an entire section of pavement on a spot-by-spot basis. Viewed this way, patching costs are considered on a lane-mile basis as long as an estimate can be made of the percentage of surface area that must be patched.

Utility curve values for both approaches are shown in Table 2-18. The second approach was adopted for this project because it is more consistent with highway agency maintenance costing methods.

2.7.2 Revised Weights. Because spot rehabilitation does not necessarily alter the riding quality of the pavement except in the vicinity of the repair work, it was considered inappropriate to include user benefits, user energy saving, and level of pavement service criteria in the decision analysis. Consequently, the weights on each of these criteria were set to zero within the computer program.

Table 2-18. PATCHING CONSTRUCTION COST

Spot Patching Cost, $ / yd^2	Patching Maintenance Cost, $ / Lane-mile	Utiltiy
4.00	100.00	1.00
12.00	500.00	0.84
20.00	800.00	0.50
110.00	2500.00	0.16
200.00	5000.00	0.00

Note:
1 yd^2 = 0.83 m^2
1 mi = 1.6 km

3. STABILIZATION OF SUBLAYERS IN-PLACE

3.1 Introduction

Stabilization of sublayers in a pavement structure has been successfully employed for many years. In fact the clay subgrade for portions of the Appian Way was stabilized with lime some 5000 years ago. Stabilization of existing soils, whenever possible, has always been an economical approach by improving the strength of natural materials rather than replacing with a stronger material.

Traditionally, three stabilizing agents have been used and are widely accepted: lime, asphalt, and portland cement (often with fly ash). Table 3-1 contains a summary of the traditional methods of stabilization (1). Patents for mixtures of soil and portland cement were issued as early as 1917 and numerous reports of investigations have been published (2-5). In the United States lime stabilization appears to have been first used around 1920 (6) and extensive use has been made of it since WW II, when Texas pioneered its present-day utilization. During the last 10 years there have been at least 300 reports published on lime stabilization (6), several of which contain excellent state-of-the-art summaries (7). Asphalt, or bitumen, is a highly successful stabilizing agent with many sands and silts (8).

These stabilization techniques have all been developed for new construction or re-construction where the stabilizing agent is intimately mixed with the natural material and compacted. For rehabilitation work this is an expensive and time-consuming process. Thus, the question arises as to methods for in-place stabilization of pavement sublayers. Such methods, if they are energy and time efficient, may be suitable even if they are more expensive.

The areas where in-place stabilization has developed the most appear to be in structural foundation work and tunnelling. An excellent state-of-the-art report has been prepared by Mitchell (9) in which he discusses methods applicable to (a) treatment of cohesionless soils and (b) treatment of fine-grained cohesive soils. Another report, by Robnett (10), summarizes a number of currently used and potential methods of stabilizing sublayers in

Table 3-1. Soil types and stabilization methods that can be used to improve stability and durability of subgrade.

Item	*Soil type*	*Possible stabilization methods*
Improve stability	Coarse granular	Compaction, soil bituminous, soil cement, cement-treated base, cement-modified soil, mechanical blending. Soil lime or lime-modified soil in soil containing reactive fines
	Fine granular	Compaction, soil bituminous, soil cement, cement-modified soil, mechanical blending, soil lime or lime-modified soil if soil contains reactive fines, or if a pozzolan (fly ash for example) is added
For silty and clayey soils. Includes reduction in resilience of elastic soils in silt and clay range (also micaceous) soils). Also prevention of erosion of subgrades of all soil types associated with pumping of portland cement concrete pavements	Very silty soils	Compaction, soil cement, cement-modified soil, and mechanical blending with or without portland cement. Soil lime when reaction is adequate for desired results
	Clayey soils of low plasticity	Compaction, soil cement, cement-modified soil, soil lime, lime-modified soil
	Highly plastic clays	Compaction, soil lime, and lime-modified soil or combinations of cement and lime. Bituminous envelopes
Control volume change (includes shrinkage and swell; also includes compaction under traffic)	Clayey soils of low plasticity	Moisture and density control to result in minimum volume change from the "as-built" to the "in-service" condition and thereafter under service conditions. Also by use of soil cement, cement-modified soil, soil lime, lime-modified soil, or mixtures involving both stabilizers. Waterproofing agents where effective.
	Highly plastic clays	Moisture and density control in lower portion of subgrade plus adequate thickness of soil lime or lime-modified soil—the degree of modification to satisfy conditions; protection with adequate bituminous envelope
	All granular soils	Compaction, to the degree needed
Frost action (includes steps to control uniformity of heaving, total heave, and control of reduction in strength during frost-melting period)	All nonuniform frost-susceptible soils	Thorough mixing—longitudinally and transversely to avoid abrupt changes in soil properties, with uniform compaction, soils of low permeability may respond to use of freezing temperature depressants (including chlorides) for extended periods. Evidence that insulation between subgrade and base may be highly effective
	Granular soils with some fines	Soil cement, cement-modified soil, soil bituminous (to fill voids), and in some instances soil lime or lime–fly ash
	Silts	Soil cement
	Clays of low plasticity	Soil cement, soil lime. Waterproofers and modifiers under favorable conditions

From Reference 5.

place. Table 3-2, from this report, lists various treatments and gives some valuable cost information.

In the following paragraphs several potential in-place stabilization techniques are briefly discussed.

3.2 Electrical Methods

Electrical Methods include electro-osmosis and electro-chemical injection. Electro-osmosis is a dewatering and strengthening mechanism resulting from the creation of an electrical gradient between positive (anode) and negative (cathode) electrodes imbedded in the soil such that charged ions move in the soil-water system. The net effect is a movement of water from the anode toward the cathode which strengthens the soil. In addition, sometimes electro-chemical effects may develop, including ion exchange, ion diffusion, mineral decomposition, precipitation of secondary minerals and physical absorption. The net result is an increased strengthening that appears to be permanent.

Electro-chemical injection is a combination of electro-osmosis and addition of stabilizing chemicals which are driven through the soil mass by the electrical gradient. Dearstyne and Newman (11) describe the successful injection of 765 gal (2900 l) of a cationic organic stabilizer (Aliquat H226, an ammonium chloride compound which acts as a waterproofer) under a 55 ft (17 m) long by 150 ft (46 m) wide section of the Seattle - Tacoma International Airport. Treatment was continued for 18 days under a potential drop of 270 v between electrodes. Robnett (10) summarizes the types of chemicals used and results obtained (Table 3-3).

3.3 Chemical Grouting

The art of grouting has been practiced over the world for the past 100 years in the construction of foundations and abutments for dams, tunnels, wells, and mine shafts. Grouting is normally defined as the injection of a fluid material into the voids of a soil mass to stop or reduce water movement, or to consolidate and strengthen the soil mass. Grouting is applicable mainly to relatively pervious soils and is normally accomplished by placing pipes in the ground, either vertically, on some angle, or horizontally at intervals, and injecting a fluid into the soil through

Table 3-2. Summary of Pertinent Characteristics of Various Subsurface Treatment Procedures

Procedure	Stability: Region of improvement	Stability: Degree of regional improvement	Stability: Ability to obtain Equivalent embankment CBR = 20	Construction: Construction treatment time	Construction: Degree of pavement disturbance	Considerations: Disruption to Runway Use	Considerations: Special Equipment Requirements	Permanency of Treatment	Amenable Soil Types	Costs
Electrical methods	Stability gradient decreasing with distance from anode, may reduce stability in cathode region)	Very low to low (possible stability reduction in cathode region)	No			Substantial	Electro-osmosis Electrical power source Dewatering electrodes Electro-chemical Electrical power source Hollow, perforated electrode Aluminum sacrificial electrodes	Some permanent strength increase	Fine-grained soils (silts and clays)	Very high (large power consumption per cubic yard)
Electro-osmosis	Stability gradient (variable with chemical type, treatment time, etc.)	Low (depends on type of chemical grout and total amount of grout used)		Very long	Moderate (depends on electrode spacing)					
Electro-chemical				Long						
Chemical grouting	Zone improvement (depends on permeability of soils)	Moderate to very high (depends on type of chemical grout and total amount of grout used)	Feasible	Moderate	Moderately high (requires relatively close grout hole spacing)	Substantial	Pressure grouting equipment; special equipment required to handle certain chemicals	Permanent	Minimum allowable grain size 0.01 mm-0.1 mm (depends on type of chemical grout)	$25-$100/ yd^3 [a] of treated material
Cement grouting	Zone improvement (depends on permeability of soils)	High to very high (depends on W/C ratio, additives and total amount of grout injected)	Feasible	Moderate	For injection Moderate For Mixed-in-place High	Substantial	For injection Pressure grouting equipment	Permanent	For injection D_{15} of soil >1.6-2.0 mm For mixed-in-place Possibly all fine-grained soils	For injection [a] $8-$15/ yd^3 of treated material
Lime procedures Drilled hole Pressure inject	Zone improvement (zone size increases with time)	Moderate to high (if soil is lime reactive)	Feasible	Moderate	Drilled-hole High Pressure injection Moderate	Substantial	For injection High pressure grouting equipment	Permanent	Any fine-grained soil with greater than 10% clay	Drilled Hole [a] $1.50-$2.50 per hole For injection $0.90-$1.80/ yd^3 of treated material
Mixed-in-place	Zone improvement	High to very high	Feasible	Moderate	High	Substantial	Special mixing apparatus	Permanent	Almost any soil type	—[b]
Post-hole-pile	Zone improvement	High to very high	Feasible	Moderate	High	Substantial	Auger	Permanent	Almost any soil type	Similar to drilled hole lime
Heat treatment	Zone improvement	High to very high	Feasible	Moderate	High	Substantial	Equipment for obtaining high temperatures	Permanent	Works best with loess soils	—[b]

[a] Multiply cost per cubic yard by 1.30 to obtain cost per cubic meter.
[b] No cost information found in literature.

From Reference 9.

Table 3-3. Types of Chemicals Used and Strength Results from Selected Electrical Treatment Laboratory Studies.

Investigator	Soil treated	Type of chemical	Strength test[a]	Post-treatment strength, in pounds per square inch[b]	Strength improvement, in pounds per square inch[b]
Adamson, et al. (1)	Montmorillonite clay	$CaCl_2$ and $Al_2(SO_4)_3$	T	cohesion = 3	1.4
Adamson, et al. (2)	sand-clay mixture	$CaCl_2$ and $Al_2(SO_4)_3$	T	cohesion = 0.8-1.4	0.8-1.4
Boiko (7)	clayey soil	$CaCl_2$ and CaO	UC	70-140	—
Esrig (15)	illite-grundite mixture	$CaCl_2$	VS	cathode = 8-11 anode = 0.2	—
Esrig (14)	clayey soil	H_2O	SS	1.1	0.2
		KOH	SS	1.1-3.1	0.3-2.2
		$CaCl_2$	SS	1.8-2.8	1.7-2.8
		NH_4Cl	SS	0.8	0.7
		KH_2PO_4	SS	1.5	1.4
		Fe SO_4	SS	0.2	0.15
		Na_4SiO_4	SS	1.4-4.7	1.3-4.6
		Na_2SO_4	SS	1.8	1.7
		$Al_2(SO_4)_3$	SS	0.5	0.4
		AM-9[c]	SS	0.4-0.7	0.1-0.6
		Arquad 2HT-75[c]	SS	0.35	0.3
		Aliquat H226[c]	SS	0.35-1.6	0.3-1.5
		Chempact[c]	SS	0.3	0.28
Karpoff (25)	sandy silt and medium clay	$AlCl_3$ and $CaCl_2$	T	cohesion = 45-53	5-13
Murayama and Mise (33)	Japanese clay	Al electrodes	T	—	0.4
Nettleton (34, 35)	kaolinite and bentonite	Al and Ca electrodes	UC	0.7-4.5	0.2-3.8
Shukla (43)	India soils	Al and Ca electrodes	T	cohesion = 4-16	1.0

[a]T = triaxial; UC = unconfined compression; VS = vaneshear; SS = shear strength.
[b]To change pounds per square inch to kilonewtons per square meter multiply by 6.895.
[c]Proprietary product.

From Reference 10.

the pipes. If soil fracture is desired, high pressures are employed. If soul fracture is to be avoided the injection pressure is kept below the overburden pressure (termed penetration grouting). Two excellent reports have been published recently which cover the fundamentals and applications of grouting in soils (12, 13). Thus this subject will be treated only briefly in this report.

The ideal grouting system is a singly Newtonian fluid with the lowest possible viscosity, a controlled setting time and an appreciable gel strength of relatively long-term performance. The cost of the system should be as low as possible. The two types of grouts in use are:

a. Particulate, or non-Newtonian grouts containing particles in suspension, such as lime, cement, and bentonite.

b. True solution or Newtonian fluid grouts, such as some chemical grouts.

The magnitude of strength increase to the sublayer that can be affected by grouting depends both on the ability to permeate the system with the grout and the pre-injection strength of the soil-water system.

There are a number of chemical grouting agents available for use (Table 3-4). The 1976 costs comparisons of these grouting materials are (12):

Cost Comparisons of Grout

Type of Grout	Basic Cost Figure
Portland Cement	1.0
Silicate Base - 15%	1.3
Lignin Base	1.65
Silicate Base - 30%	2.2
Silicate Base - 40%	2.9
Urea-formaldehyde Resin	6.0
Acrylamide (AM-9)	7.0

The 1976 cost per yd^3 (multiply by 1.3 for cost per m^3) of soil grouted with chemical grout ranges from $40 to $190 (12).

The range of soil grain sizes over which different grouts have been successfully used is given in Figure 3-1 (9).

To successfully high-pressure inject a grout into a sublayer, channels and passages must either exist or occur as a result of the pressure. In this method, a particulate grout is normally used and the resultant stabilized system is a non-homogenous, stratified system of natural material with

Table 3-4. Properties of currently used grouts.

GROUT MATERIAL	CATALYST MATERIAL	UNCONFINED COMPRESSIVE STRENGTH (PSI) OF GROUTED SOIL	VISCOSITY (CENTIPOISE)	SETTING TIME MINUTES	TOXICANT*	POLLUTANT**
SILICATE BASE						
LOW CONCENTRATION	BICARBONATE	10-50	1.5	0.1 - 300	NO	NO
LOW CONCENTRATION	HALLIBURTON CO. MATERIAL	10-50	1.5	5 - 300	NO	NO
LOW TO HIGH CONCENTRATION	SIROC - DIAMOND SHAMROCK CHEMICAL CO.	10-500	4-40	5 - 300	NO	NO
LOW TO HIGH CONCENTRATION	CHLORIDE - JOOSTEN PROCESS	10-1000	30-50	0	NO	NO
LOW TO HIGH CONCENTRATION	ETHYL ACETATE SOLETANCHE & HALLIBURTON	10-500	4-40	5 - 300	NO	NO
LOW TO HIGH CONCENTRATION	RHONE-PROGIL 600					
LOW TO HIGH CONCENTRATION	GELOC-3 H. BAKER CO.	10-500	4-25	2 - 200	NO	NO
LOW TO HIGH CONCENTRATION	GELOC - 3X	10-250	4-25	0.5 - 120	NO	NO
LIGNIN BASE						
BLOX-ALL	HALLIBURTON CO. MATERIAL	S-90	8-15	3 - 90	YES	YES
TOM	CEMENTATION CO. MATERIAL	50-500	2-4	5 - 120	YES	YES
TERRA-FIRMA	INTRUSION CO. MATERIAL	10-50	2-5	10 - 300	YES	YES
LIGNOSOL	LIGNOSOL CO. MATERIAL	10-50	50	10 - 1000	YES	YES
ACRYLAMIDE BASE						
AM-9 ***	DMAPN and AMMONIUM or SODIUM PERSULFATE	50-500	1.2 - 1.6	0.1 - 1000	YES	YES
FORMALDEHYDE BASE						
UREA-FORMALDEHYDE	HALLIBURTON CO. MATERIAL	OVER 1000	10	4 - 60	YES	YES
UREA-FORMALDEHYDE	AMERICAN CYANAMID CO. MATERIAL	OVER 500	13	1 - 60	YES	YES
RESORCINOL FORMALDEHYDE	CEMENTATION CO. MATERIAL	OVER 500	3.5	—	YES	YES
TANNIN - PARA-FORMALDEHYDE	BORDEN COMPANY MQ-8					
GEOSEAL MQ-4 & MQ-5	BORDEN COMPANY MATERIAL					
UNSATURATED FATTY ACID BASE						
POLYTHIXON FRD	CEMENTATION CO. MATERIAL	OVER 500	10 - 80	25 - 360	NO	NO

* - A material which must be handled using safety precautions and/or protective clothing.
** - Pollutant to fresh water supplies contacted.
*** - Also available from grouting companies under trade names of PWG or Injectite-Q.

From Reference 12.

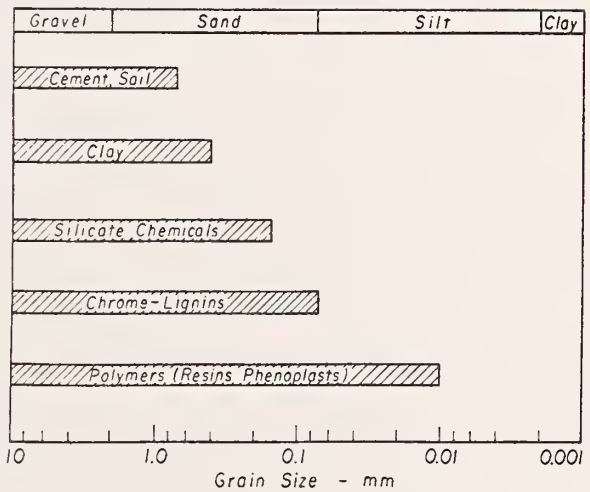


Fig. 3-1 - SOIL GRAIN SIZE RANGES OVER WHICH DIFFERENT GROUT TYPES ARE USEFUL.

From Reference 9.

lenses and layers of grout. The biggest problem with this method is the potential for loss in surface profile and grade as the pressure uplifts the pavement. Thus, this method offers very little potential in pavement rehabilitation.

Successful low pressure, or penetration, grouting depends upon the permeability of the sublayer to be stabilized. Probably the greatest limitation to this method is the inability to penetrate fine-grained sublayers with the grout. Karol (14) estimates the radial distance a grout may be penetrated from an injection point is:

$$r = 0.62 \sqrt[3]{\frac{Rgt}{n}}$$

(Multiply radial distance by 0.3 to convert to meters) in which r = radial distance in ft; R = ratio of water-to-grout viscosity; g = rate of grout take, in ft^3 per min; t = gel time, in min; and n = soil porosity.

Johnson (15) has demonstrated a useful criterion for determining the penetration groutability (GR) with particulate grouts.

$$GR = \frac{(D_{15})\ \text{Soil}}{(D_{85})\ \text{Grout}} > 20 - 25 \text{ for success}$$

in which D_{15} = 15% finer particle size of the sublayer; and D_{85} = 85% finer particle size for the particulate matter in the grout.

For successful particulate grouting of soils consisting of predominately one grain size, the grout particles should be less than one-tenth of the soil particle mean size (10).

The in-place cost (1976) for soil grouted with cement is approximately \$13.50 to \$35.00 per yd^3 of soil grouted (multiply by 1.3 for cost per m^3) (12).

3.4 High Strength Post-Hole Piles

The post-hole pile method, pressure injection lime method, and the drilled hole lime method of stabilization has been employed in a test section at Altus AFB, Oklahoma (16). In this method--a system which places high-strength, vertical inclusions in the subgrade beneath the pavement at relatively close spacings--surface deflections and vertical compressive stresses are significantly reduced (17). In the Altus test

sections, these piles were 1.0 ft (0.3 m) in diameter, 7 ft (2.1 m) long, and spaced 3 to 5 ft (0.9 to 1.5 m) apart. The runway pavement structure consisted of a 4 in. (10 cm) asphaltic concrete surface, a 6 in. (15 cm) crushed stone base, and 22 in. (56 cm) of subbase over a clay subgrade (AASHO classification A-6). The piles were of two materials, a 3000 psi (20,700kPa) concrete, and a 1000 psi (6900 kPa) lean - concrete. Construction occurred in July - August 1973. Preliminary results utilizing a simulated C-5A aircraft loading indicate the strength of the pile material was not a significant factor.

From Table 3-2, the cost (1973) of this post-hole pile system should range from $1.50 to $2.50 per hole for injection.

3.5 Heat Treatment

Heat treatment offers a way whereby weak, highly plastic clays might be rendered inert and quite strong. This is not a new concept. Bricks, lightweight aggregates, and other synthetic aggregates have been made from clay for many years. Highly plastic clays, notably those comprising significant quantities of kaolinite and illite, can be effectively transformed from clays into essentially amorphous silicates at moderate (500°C) to high temperatures (1000°C) and reasonably short times (10, 18). The transformation process is quite complex involving non-equilibium thermodynamic phase changes, and the effects of time and temperature are inversely related. Thus:

$$D \propto \frac{T}{t}$$

Where: D = degree of thermal transformation
T = temperature
t = time

Lambe (19) reports that an expansive clay, having a swelling index of 0.11 after heating to 105°C had its expansiveness reduced to a swelling index of only 0.03 after heating to 750°C. Moore (20) reports:

> 1. The clay minerals, montmorillonite, illite, and kaolinite, will not rehydrate under atmospheric conditions once they have been completely dehydrated (dehydroxylated); therefore, once they have been completely dehydrated, they become chemically stabilized for use

as highway construction materials. Complete dehydration is accomplished by heating the clay and holding it at the elevated temperature for sufficient time to allow the dehydration to occur. A period of 15 minutes at 1400°F. was sufficient to completely dehydrate the clay present in the small, oven dry laboratory specimens made from the Texas soils investigated.

3. Most (if not all) clay-type soils having a relatively high strength when air dried can be fired to produce hard, durable aggregates suitable for use in flexible base and asphaltic concrete.

Heat treatment has also been attempted to stabilize a collapsable loessial soil. Litvenov (21) reports that typical burn holes spaced three ft (1 m) on center were heat treated at moderate temperatures for 8 to 10 days to stabilize the loess.

Burners to provide heat into holes drilled in the soil mass can be found from among several burner manufacturers. One such manufacturer, The Hauck Manufacturing Co., P. O. Box 90, Lebanon, Pa. 17042, makes a wide variety of burners, including the CA10 gas-oil burner which appears to be suitable for heat treatment of soils.

3.6 Current Stabilization Equipment

A general summary of special equipment requirements is given in Table 3-2. For electrical methods a tabulation of materials and costs is given in Table 3-5 (21). These costs were for stabilizing a 200 ft (60 m) long by 40 ft (12 m) wide by 3 ft (1 m) deep volume of Chinle clay in Arizona. This clay is classified as an A-7-5 according to the AASHO System and the stabilization is for the purpose of reducing the expansive potential of the clay from moisture changes.

For chemical grouting, the types of injection pipes are given in Table 3-6. Injection pipes, together with a mixing and pumping system, comprise the overall system. Most contractors use either positive displacement or progressive cavity type pumps (normally one pump per hole) and pump a batched mixture of grout.

Boring vertical holes in the ground for grouting or piles is an easy task using one of the many types of boring machines. However, boring horizontal holes is another matter as horizontal augers, or borers, are not so common. One horizontal auger manufacturer is McLaughlin Manufacturing

Table 3-5. ELECTRO-OSMOTIC TREATMENT
Materials and Costs (1969 data)

Item and Description	Quantity	Unit Cost	Total Cost	Recoverable After Testing
9 Anodes, 6" dia., 1/4" wall x 20' long, 2' riser on each end (see Fig. 66a)	216 ft	$2.00/ft	$432.00	yes
36 Cathodes, No. 6 Rebar, 12' long (see Fig. 66b)	432 ft	$0.10/ft	43.20	yes
18 Resistivity bars No. 6 Rebar, 12' long (see Fig. 66c)	54 ft	$0.10/ft	5.40	no
54 Ground clips (used to connect wiring to cathodes and resistivity bars)	54	$0.25 each	13.50	yes
No. 00 wire (used to wire anodes into circuit	60 ft	$1.00/ft	60.00	yes
No. 12 wire (used to wire cathodes into circuit)	3500 ft	$0.55/ft	175.00	yes
No. 20 wire (used to wire resistivity bars into circuit)	2000 ft	$0.08/ft	16.00	no
1/2" dia. garden hose	700 ft	$0.04/ft	28.00	yes
Current Monitor Panel	1	none	none	yes
Voltage Monitor Panel	1	none	none	yes
62 KW Generator	1	none	none	yes
Generator Fuel	1000 gal	$0.15/gal	150.00	no
5000-gallon solution tank	1	$525.00	$525.00	yes
Potassium chloride (muriate of potash)	12,000 lb	$0.027/lb	$325.00	no
TOTAL COST OF NON-RECOVERABLE ITEMS			$496.40	
LABOR (Installation, maintenance, removal--six men, one week)			$600.00	
				$1096.40*
TOTAL COST OF RECOVERABLE ITEMS			$1276.70	$1276.70
TOTAL COST OF TREATMENT--200-foot section				$2373.10

*NOTE: This represents a cost of $5.432 per lineal foot of treated section.

1 inch = 25.4 mm, 1 foot = 0.305 m, 1 gal = 3.785 l, 1 lb = 0.454 kg

From Reference 21.

Table 3-6. Types of Injection Pipes

NAME	DESCRIPTION	PLACEMENT	ADVANTAGES	LIMITATIONS
A. Drive Rod (Lances)	EW Rod with special pointed end or extrudable plug.	Driven in ground	Pipe retrieved. Can grout in zones or stages.	For depths to about 50 feet. Requires driving equipment
B. Slotted Pipe	Plastic pipe with slots.	Set in borehole w/gravel & grout.	No special equipment required	Can grout only one zone.
C. Tube à Manchette (See Fig. 18)	Plastic pipe with sleeve covered holes at given intervals (French origin, available in Europe).	Set in borehole with weak grout. Uses inner pipe with packers to straddle holes.	Can grout selectively and regrout as desired.	May not be available in the United States
D. Lost Injection Element	Single section element with sleeve covered holes (Netherlands)	By special steel tube with small plastic hose to surface. Element is left in ground.	Simple if contractor has equipment required.	May not be available in U.S. Special placement vibrator crane is required. Can only grout one section about 1 meter thick.
E. Open Hole With Packer	Uses cased hole with air packer set in end of casing above zone to be grouted.	In borehole.	Can stage grout as desired.	Requires packer and air source.
F. Stabilator Valve Tube	Steel drilling system also used for grouting (Swedish).	Drills with pipe, then knocks off bit and uses pipe for grouting.	Can grout selectively.	May be hard to obtain in the United States.

1 ft = .30 m

From Reference 13.

Co., P. O. Box 303, Plainfield, Illinois 60544, who manufacture the Western Boring Systems for horizontal borings (Figure 3-2). These systems can bore holes from 3 in. to 30 in. in dia (8 to 80 cm).

3.7 Utility of Potential Rehabilitation Techniques

During the course of investigation on this project some techniques were conceived and others were found in the literature. Those not considered to offer any potential advantages over existing techniques were discarded during the preliminary screening process (see Appendix B). The remaining techniques appear to have potential in one or more particular situations. From the preceding discussion it is apparent that in-place stabilization is quite expensive and thus has been considered in only those situations where:

1. Rehabilitation time needs to be as short as possible, or
2. Only a localized failure from a weak sublayer needs repair.

Table 3-7 summarizes the applicability of each potential rehabilitation technique to solve specific problems in terms of each technique's overall utility for certain assumed conditions; to wit:

1. Urban freeway with a volume/capacity ratio = 1.0
2. Serviceability index at time "0" is 2.9 before rehabilitation and 4.2 after rehabilitation (which includes a surface dressing)
3. Without rehabilitation the serviceability index reduces to 2.5 in four years and remains at 2.5 thereafter
4. Skid number onitted

The overall utilities of each technique to solve particular problem is only as valid as the input assumptions. Therefore, every attempt was made to obtain accurate and realistic values for all the decision criteria. In some cases such data were not available and an "educated guess" was made to estimate these values.

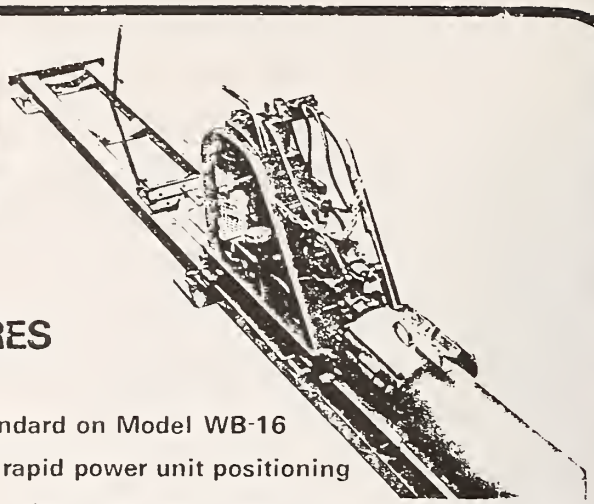
Examination of the utility values in Table 3-7 reveal:

1. For granular, porous, subbases too weak, current techniques of overlaying appear to be the best, although high pressure PC grout is almost as good for rigid pavements.

2. For highly plastic clay subgrades too weak, current techniques of overlaying appear to be the best.

WESTERN Boring Systems

MODEL WB-16



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SPECIFICATIONS

GENERAL

Engine	4 cycle gasoline powered, 12 hp.
Starter	12 volt electric
Transmission	Hydro-Static
Final Drive	Two-stage gear reducer
Hydraulic System	2000 psi
Thrust	14,000 lbs. std.
Auger Drive	1⅝" hex
Auger Size	Without casing 3" - 16"
	With casing 4" - 16"
Bore Centerline	10"

BORING CHARACTERISTICS

Auger Speed	0-60 rpm, forward or reverse
Maximum Torque	1300 ft. lbs. (approximate)

CAPACITIES

Fuel Tank	1.5 gal.
Hydraulic System	5 gal.

DIMENSIONS

Carriage	4'-0"long x 1'-10"wide x 3'-6"high Mounted height 3'-10½"
Master Track	10'-2"long x 2'-7"wide x 4½"high
Track Extension	10'-0"long x 2'-7"wide x 4½"high

WEIGHTS

Carriage	950 lbs.
Master Track	350 lbs.
Track Extension	338 lbs.

BORING DISTANCE IN CASING

DISTANCE VARIES WITH SOIL CONDITION

CUTTING HEAD DIA. (IN.) ↓	MAX. DISTANCE (FT) → 60	70	80	90	100	110	120	130	140	150
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12										
10										
8										

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Figure 3-2. Horizontal Boring System

1 inch = 25.4 mm, 1 PSI = 6.89 kPa, 1 lb = 4.45 N, 1 gal = 3.785 l, 1 foot = 0.305 m

Table 3-7. Utility Values for In-Place Stabilization Techniques to Solve Specific Problems.

Technique	Granular, Porus Subbase too Weak		Highly Plastic Clay Subgrade too Weak		General PCC Failure		Localized Sublayer Failure	
	Flexible	Rigid	Flexible	Rigid	Joints	Edge	Flexible	Rigid
A. Potential Techniques								
Electro-Osmosis plus 1 in. (2.5 cm) AC overlay			0.654	0.644			0.706	
Electro-Chemical plus 1 in. (2.5 cm) AC overlay			0.652	0.650			0.702	
High Pressure PC Grouting		0.740			0.740	0.740		0.651
Penetration Chemical Grouting	0.634					0.730		
High Strength Post-Hole Piles	0.568		0.568				0.633	
Heat Treatment of Subgrade				0.569				0.511
B. Present Techniques								
2 in. (5 cm) AC overlay	0.728	0.753	0.728	0.753	0.737		0.682	0.682
Dig Out and Replace					0.686		0.706	0.650
Seal with Polysulfide (see chapter 6)					0.676	0.676		

3. For PCC joint failures, high pressure PC grouting appears to be about as good as a 2 in. AC overlay.

4. For PCC edge failures, high pressure grouting and penetration chemical grouting appear very promising.

5. For localized sublayer failures in flexible pavements, both electro-osmosis and electro-chemical injection appear to be about as good as the current techniques of dig out and replace.

6. For the stated conditions, neither high strength post hole piles nor heat treatment of the subgrade appear promising.

Thus, some techniques show potential and others do not. The techniques recommended for further consideration are summarized in Table 3-8. Note they are recommended for specific conditions of distress, pavement type, and traffic. Due to the ever present constraint of time and money only selected conditions were examined. These conditions were chosen so as to maximize the potential usefulness (or value) of each particular technique. Therefore, when a technique's value comes out low (such as the subgrade heat treatment technique), we may generalize and conclude that--under present circumstances--such a technique would not show promise under almost any condition. But, the reverse is <u>not</u> true. Those techniques which indicate promising potential, under certain conditions, may or may not appear promising under different conditions. Their potential use under different conditions would have to be evaluated separately.

The use of utility decision analysis has resulted in the researchers being able to compare different techniques in a quantitative and rational manner--certainly one of the most valuable outputs of this research project.

Table 3-8. Summary of Promising In-Place Stabilization Techniques and Applicable Conditions

Potential Technique	Applicable Conditions
High Pressure PC Grouting	Urban freeway, rigid pavement, with either (1) joint or edge faulting and pumping or (2) porous subbase too weak.
Penetration Chemical Grouting	Urban freeway with either (1) flexible pavement over a porous subbase too weak or (2) rigid pavement with edge pumping.
Electro-Osmosis plus 1 in. (2.5 cm) AC Overlay	Urban freeway flexible pavement, with a localized failed area due to a weak clay subgrade.
Electro-Chemical plus 1 in. (2.5 cm) AC Overlay	Urban freeway, flexible pavement, with a localized failed area due to a weak clay subgrade.

4. SURFACE REHABILITATION OF FLEXIBLE PAVEMENTS

4.1 Introduction

The idea of renewing the serviceability of a flexible pavement by direct correction of surface defects, in lieu of using overlays, is quite attractive because this approach appears to offer a simple, quick, and inexpensive solution to many pavement deterioration problems. In fact, surface treatments such as the application of aggregate seal coats, "chip seals", have been in use for some time. However, in the context of this study, the use of chip seals is not considered to be innovative. Additionally, it should be noted that any rehabilitation technique that only treats the pavement surface layer will not be useful in relieving certain kinds of pavement distress because such methods will be only marginally successful if pavement deterioration is the result of an inadequate pavement structure.

Surface rehabilitation techniques are useful primarily for alleviating the following kinds of flexible pavement distress:

1. Aging and surface hardening resulting in loss of resiliency and water resistance.
2. Loss in surface macrotexture and microtexture resulting from raveling.
3. Distortion of the normal road surface profile and cross-section (e.g. rutting) not the result of sub-surface failure.
4. Surface cracking, not resulting from fatigue failure or otherwise attributable to weak sub-layers.
5. Asphalt-rich slick surfaces resulting from bleeding or flushing.
6. Random surface defects (holes) such as pot holes, indentations and the like.

The surface rehabilitation techniques proposed in this study to solve such pavement distress problems can be grouped into the following categories.

1. Mechanical manipulation with or without simultaneous direct application of heat.
2. Surface rejuvenation by treatment with special materials (chemicals).

3. Surface rejuvenation by physical methods.
4. Reworking or replacement of part or all of the pavement riding surface layer, usually limited to a depth no greater than 2 in. (5 cm).

Since application of many of these techniques will not, by themselves, result in a smooth, skid-resistant riding surface, a final finishing treatment or surface dressing sometimes must be added to maximize the potential benefits to be gained. Accordingly, where necessary for this purpose, the complete process of applying a given technique may include a chip seal or thin overlay.

4.2 Mechanical Surface Manipulation

Pavement rehabilitation methods in this category involve in-place reworking of the pavement surface to a shallow depth (usually less than 1.5 in. or 4 cm) with or without direct application of heat. The methods include hot and cold, planing and milling. These techniques are particularly useful for correcting distortion of the road profile or cross-section caused by rutting, shoving and the like. At the same time the methods can be applied so that there is no increase in the elevation of the riding surface. Another approach to surface manipulation involved heating the road surface to a plastic state followed by recompaction.

After such mechanical manipulation procedures are applied, a dressing for the riding surface usually will be necessary to produce a smooth-riding, skid-resistant pavement. This sometimes can be accomplished by in-place reprocessing and relaying any material removed or by spreading and imbedding a layer of polish-resistant aggregate into the old asphalt-aggregate surface while it is still warm. However, better results normally can be obtained by using a chip seal or thin overlay as a surface finish dressing.

Heater-planers were among the first (around 1930) types of equipment used for highway surface in-place reworking. Early models were primitive, consisting of a heating unit followed by an ordinary motor grader. Currently available equipment is specifically designed for the job (1, 2, 3, 4, 5, 6), and may combine both heating and planing operations (Cutler and Jim Jackson) or may comprise two pieces of equipment (Payne).

A summary of heater-planer equipment features, costs, and energy requirements prepared by Epps (7) is given in Table 4-1.

A cold planer has been developed very recently (8) which has the potential of avoiding some of the hot planer problems such as pavement overheating and air pollution. The machine employs hydraulically operated vibrating beams fitted with cutting edges. It is capable of removing 1 ½ to 2 in. (3 to 5 cm) of bituminous surface over a 6 ½ ft (2 m) width at a rate of 50 ft (15 m) per min. Estimated fuel consumption is 70 gal (265 dm^3) per 8-hour day which indicates 0.004 gal/yd^2 (0.018 dm^3/m^2) equivalent to 560 Btu/yd^2 (2.7 x 10^6 J/m^2).

Rutting, bumps and depressions also can be effectively removed by use of rotary cutters; that is, by a kind of traveling milling machine. Equipment is available for milling the road surface in either a heated or unheated condition (9, 10, 11, 12). Even with hot-millers the pavement is worked at a much lower temperature (120 to 150°F or 49 to 66°C) than with heater-planers (300 to 400°F or 149 to 204°C). Thus the application of such equipment appears to be a way of saving energy, reducing air pollution, and avoiding pavement damage when material is removed from a road surface. Road rehabilitation techniques involving surface milling have not been applied extensively in this country. Although a number of field trials have been completed (5, 13, 14) some allowance in time and money for further development, or at least field trials, should be made. However, results to date are promising. One study (5) indicated potential surface removal depths of 1.5 in. (4 cm) for a single pass, compared with a 3/8 in. (1 cm) average in heater-planer operation.

This study also confirmed energy savings but the benefits with respect to reduction in air pollution appear to be mixed. Summaries of hot-milling and cold-milling equipment features, costs, and energy requirements are given in Tables 4-2 and 4-3.

Another approach to flexible pavement rehabilitation by mechanical manipulation of the surface involves heavy rolling. This method is most suited to removal of the distortion of the road profile or cross section (e.g., rutting) caused by residual deformation in any of the flexible pavement layers. In most cases it will be necessary to complete such

Table 4-1. Summary Comparison of Heater-Planer Equipment.

Manufacturer	Equipment Features	Costs		Fuel				Comments
		$/yd^2 ($/m^2)	Includes	Type	Quantity gal/yd^2 (dm^3/m^2)	Btu/yd^2 J/m^2	Includes	
Asphalt Equipment, Inc.	heats and planes	0.15-0.28 (0.18-0.33)	equipment rental, fuel and labor to heat and plane	propane				fire horizontally 3/4 in. (2 cm) removal
Cutler Helio-Planer	heats, planes and elevates	0.50 (0.60)	equipment rental, fuel, labor and elevation of material	propane	0.2 (0.9)	18,200 (2.3×10^7)	heating power unit	indirect heating 250 yd^2 (209 m^2) per hr. 3/4-1 in. (2-2.5 cm) removal
				diesel	0.02 (0.09)	2,800 (3.5×10^6)		
Jim Jackson Heater-Planer				propane		10,100 (1.3×10^7)	heating	direct heating 1,000 yd^2 (836 m^2) per hr. 3/4 (2 cm) in. removal
Greenside MK III Road Planer	heats and planes							
Littleford*	heats and planes							
Payne Heater-Planer	heating unit separate from planer	0.40-0.60 (0.48-0.72)	heat, plane, clean-up haul traffic control	diesel	0.15 (0.68)	20,000 (2.5×10^7)	heating and power unit	direct heating 2,500 yd^2 (2090 m^2) per day. 3/4 (2 cm) in. removal

*Equipment no longer manufactured.

Table from Epps (7)

Table 4-2. Summary-Comparison of Hot Milling Equipment.

Manufacturer	Equipment Features	Costs			Fuel			Comments
		$/yd^2 ($/m^2)	Includes	Type	Quantity gal/yd^2 (dm^3/m^2)	Btu/yd^2 (J/m^2)	Includes	
Millars Mark II Road Razer	heats, planes and elevates	1.10 (1.32) for in.(2.5 cm) cut		LPG	0.05 (0.23)	4,600 (5.8×10^6)	heating power unit	rotary cutter 1.5 in. (4 cm) removal
				diesel	0.025 (0.11)	3,500 (4.4×10^6)		
Wirtgen F800/1000	heats and planes			LPG				rotary cutter small unit
Wirtgen FM2100B	heats and planes			LPG	0.1 (0.45)	9,000 (11.4×10^6)	heating	rotary cutter 1,000 yd^2 (836 m^2) per hr. 2.4 in. (6 cm) max. depth

Table from Epps (7)

Table 4-3. Summary-Comparison of Cold Milling Equipment.

Manufacturer	Equipment Features	Costs		Fuel				Comments
		$/yd^2 ($/m^2)	Includes	Type	Quantity gal/yd^2 (dm^3/m^2)	Btu/yd^2 (J/m^2)	Includes	
CMI Auto-Grade Roto-Mill	planes and elevates	0.60-1.00 (0.72-1.2)	milling and elevation of material	diesel	0.007 (0.032)	1,000 ($1.3x10^6$)	milling power unit	110 in. (2.8 m) cutting width 2000 Yd^2 (1670 m^2) per hr. 1 in. (2.5 cm) removal
Cutler Cager Beaver	planes							mill to 2 in. (5 cm) depth at 12 to 18 ft (0.37 to 55 m) per minute
Galion BP-30	planes	0.05 (0.06)	bits and fuel					31 in. (78 cm) cutting width
BJD Miniplaner	planes							12 in. (30 cm) cutting width
Payne Pavement Planer	planes	0.35-0.55 (0.42-0.66	milling, cleaning, hauling & traffic control to 1 in. depth	diesel	0.015 (0.068)	2,200 to 2,500 ($2.8x10^6$) to $3.2x10^6$)		60 in. (.15 m) cutting width 250-300 Yd^2 (209-250 m^2) per hr. 1 in. (2.5 cm) removal
Payne Mini-Planer	planes			diesel				12 in. (30 cm) cutting width small unit

Table from Epps (7)

treatment with a light surface dressing which will also help correct cracking, surface loss and general surface deterioration.

When such rolling is applied to eliminate rutting resulting from failure of the base courses or subgrade it should be expected that the correction will be temporary if the same application of wheel loading is continued. On the other hand, correction of distress originating in the surface layer by this technique should be longer lasting. In either case, application of the heavy rolling technique will be successful only under suitable combinations of the influencing intrinsic and extrinsic variables.

These variables are somewhat different for base course or subgrade distortion correction than for surface course rehabilitation, as indicated by the following:

Application	Intrinsic Variables	Extrinsic Variables
Rolling to correct distortion originating in base courses or subgrade.	Soil classification bearing values (as indicated by CBR, R-Value, etc.). Stiffness (as indicated by Resilient Modulus).	Layer Temperature, Layer Depth and Thickness, Layer Moisture Content, Roller Characteristics (Drum width, weight and spacing, net roller weight, forward speed, frequency and amplitude of vibrating drums, number of roller passes).
Rolling to correct distortion originating in surface course.	Bituminous mix stiffness (as indicated by Marshall stability, stabilometer value, etc.). Temperature susceptibility (as indicated by binder penetration index or viscosity-temp. slope).	Layer Temperature, Layer Thickness, Roller Characteristics

Information from which the proper values of intrinsic and extrinsic variables can be inferred for rolling to correct distortion originating in the base courses or subgrade can be formed from reports by Edris and Lytton (15), Townsend and Chisholm (16), and Dempsey (17). Information including criteria for equipment selection and operation in rolling surface courses is given in a report by Nittinger (18).

An example of the application of the heavy rolling technique is one in which surface course defects are rolled out after the surface is heated to a plastic state. Uniform surface heating over a large area while avoiding local overheating is a requirement of the technique. The procedure proposed for doing this involves the following steps: (1) uniformly spreading a layer of heated aggregate (350°F or 177°C) on the pavement surface, (2) after allowing sufficient time for heat to be transferred to the pavement, removing the aggregate, and (3) immediately following the aggregate removal, roll the surface with heavy vibrating or other rollers. No literature describing field trials of this technique has been found. However, it is clear that the technique could be implemented without requiring special equipment. In fact, except for the rock heating step, conventional flexible pavement surface construction and maintenance equipment (e.g., aggregate spreaders, loaders, and heavy rollers) can be used. Construction costs and energy requirements can be based on experience in operating such equipment. Also, the technique should be relatively easy to apply. Other benefits include potentially low energy and materials requirements as well as few environmental problems if clean aggregate is used. On the other hand, unless a surface dressing or chemical treatment is applied at the same time, the service life of pavements rehabilitated in this manner is uncertain, except where the technique is used to correct defects in relatively new flexible pavement surfaces. Also, field development work will be necessary to work out the details of this technique. Particular items to be investigated include the timing and temperatures for the pavement heating stage as well as equipment selection and operating criteria for the rolling sequence.

4.3 Surface Rejuvenation by Treatment with Special Materials

Techniques in this category are generally directed toward the restoration of pavement performance believed to have been lost as a result of age hardening of the bituminous binder along with the sealing of surface openings caused by weathering and temperature cycling. Hardening of bituminous binders (as evidenced by substantial increases in viscosity and decreases in penetration values) begins during pavement construction operations and usually progresses at a decreasing rate over the life of a flexible pavement surface course.

For example, it has been reported (19) that an 85/100 penetration asphalt may decrease to 60 to 65 penetration during the normal asphalt concrete hot-mix process. As the pavement ages, further decreases in binder penetration and corresponding increases in viscosity depend on the extent of exposure, the largest changes occuring near the pavement surface. Reductions in asphalt penetration from 85/100 to values ranging from 16 to 27 in 30 years have been indicated (19). Vallerga (20) reported comparable changes in binder penetration after only 2 years service. Traxler (21) indicated reductions in the penetrations of 85/100 grade asphalt cements from eight different producers to values ranging from 26 to 40 and corresponding 6 to 23 fold increases in viscosity after one year of road exposure. Jones (22) showed reductions of penetration to values between 20 to 55 in 10 years, depending on the percent air voids in the asphalt concrete surface course.

Binder hardening may have beneficial as well as harmful effects on performance of flexible pavement surface courses. Numerous studies reviewed by Epps and Monismith (23) clearly indicate that stiffening of bituminous mix surface courses tends to increase fatigue life (and thus reduce the tendency of the road surface to develop alligator cracking). Such stiffening may also tend to reduce pavement rutting, bleeding, and abrasion losses, although these effects are not well documented. On the other hand, studies considered by Phang and Anderson (24) clearly indicate that the tendency to produce transverse cracks and associated longitudinal cracks increases as the bituminous binder hardens. Other detrimental effects of binder hardening include reduction in binder-aggregate adhesion with resulting road surface raveling and development of small fissures and cracks. On balance, it appears that the initial net result of binder hardening is favorable, but ultimately the harmful effects predominate. Dellert (25) reported that when the penetration of asphalt recovered from surface courses reached 15, excessive random surface cracking was observed on low volume roads (ADT 3000) but satisfactory performance was noted on high volume roads (ADT 30,000) with asphalt of comparable consistency.

A number of flexible pavement surface rejuvenation techniques depend mainly on softening the bituminous binder. Since the benefits of the

resulting decrease in the stiffness of the surface layer might not outweigh the disadvantages, it is clear that such treatments should be applied with judgement and caution. However, some materials, such as sprayed-on elastomer emulsions (26) result in favorable significant modification of the binder composition, so that treatment with such agents will have effects that cannot be measured solely by how they alter the surface course stiffness. In fact, this kind of binder modification has been found useful in upgrading the performance of newly constructed bituminous mix surface courses (26).

Another consideration is how well the rejuvenating materials penetrate the surface layers. A paper by Olivier (27) concludes that rejuvenation of old bituminous roads by spray treatment with commercial oil fraction penetrants will take place slowly. This conclusion is confirmed by field studies reported by Brown and Johnson (28) that encountered such problems with penetrants containing high viscosity residues but showed adequate surface absorption of water emulsions containing relatively low viscosity active components. Additionally, rejuvenating penetrants tend to reduce surface macrotexture and skid resistance (28, 29), at least for a time after their application. Accordingly, after a surface is sprayed with a rejuvenating agent, additional surface treatment such as a chip seal with skid resistant cover aggregate might be desirable.

A comparison of five different bituminous pavement rejuvenating materials that have been actively promoted is given in Table 4-4. While all of these materials are intended to favorably modify the properties of aged bituminous binders, a diversity of chemical approaches is represented. Clearly, still other chemical modifications of hardened bituminous binders are possible and this could be a fruitful field for product development. Any product offered for bituminous pavement rejuvenation should comply with the following requirements:

1. The material should be capable of being applied easily and inexpensively to a pavement surface and should quickly penetrate to a depth such that significant modification of the pavement surface layer can take place.
2. The material should be compatible with aged (oxidized and polymerized) bitumins and produce beneficial modifications of

Table 4-4. Comparison of Several Flexible Pavement Rejuvenating Materials

Material Type	References	Application Temperature °F (°C)	Potential for surface Penetration	Recommended Application Rates gal/Yd^2 (dm^3/m^2)	Estimated Total Cost of Application $/Yd^2 ($/m^2)
Emulsions of Coal-Tar Fractions	(30)	150(65)	Good	.05 to.2(.23 to.9)	0.15 to 0.24(0.18to0.29)
Cationic Emulsions of Asphalt Maltenes	(19) (27) (29) (31) (32) (33)	Ambient	Good	0.1 to 0.2 (.45 to.9)	0.15 to 0.03 (0.18 to 0.36)
Emulsions of Elastomeric Polymers	(25) (26) (34)	Ambient	Good	0.1 to 913(.45 to 1.35)	0.35 to 0.95(0.42 to 1.14)
Cut-Back Gilsonite	(21) (28) (35) (36)	Ambient	Poor	0.1 to 0.15(.45 to.68)	0.15 to 0.20 (0.18 to0.23)
CRS-1 Asphalt Emulsion	(28)	Ambient	Poor	0.1 to 0.2 (.45 to 0.9)	0.06 to 0.11(0.07 to 0.13)

road performance which outweigh any detrimental effects.

3. In the quantities required, the total cost of applying the treatment should be reasonable.
4. The material must not be hazardous to apply (toxicity and flammability) and should not result in environmental contamination.

Among the alternate materials proposed for this purpose are the lignins produced from sulfite waste liquor. These have already seen some application for soil stabilization, but no record of their application for bituminous surface rejuvenation could be found.

An unequivocal statement regarding the effectiveness of road rejuvenating compounds cannot be made. As might be expected, promotional literature, including articles in trade journals, claim unqualified success and give little indication of the limitations previously discussed. Other reports, notable references (28) and (29), do consider such problems but also indicate a potential for net benefits. Thus, it appears that use of surface rejuvenating chemicals should be seriously considered, but only after careful evaluation for a specific application by well documented field trials.

A variant of the surface rejuvenation techniques is the application of emulsion slurry seal coats. Proponents of this approach claim significant rejuvenation of the treated pavement surface resulting from penetration of the emulsion into surface openings (37). Successful crack filling is also indicated (38). However, cases of poorly controlled application of slurry seals has given this pavement rehabilitation technique a poor reputation in some quarters. Costs are quite reasonable, in the range of 0.26 \$/yd^2 to 0.50\$/yd^2 (0.31 \$/m^2 to 0.60 \$/m^2). The problem of lowered skid resistance when the agents listed in Table 4-4 are applied is avoided in slurry seal applications. In fact, improved low speed wet skid resistance can be demonstrated (39). However, the low macrotexture of slurry seal treated surfaces will tend to result in relatively low skid numbers at high vehicle speeds. The problem could be alleviated by the use of larger size gap-graded aggregates in this technique. Another potentially effective variation appears to be the

use of modified emulsions in the slurry seal technique. For example, rejuvenation of the existing surface could be enhanced by incorporating one of the more effective agents listed in Table 4-4. Emulsion additives also could be used to increase the service life of slurry seals. An exploratory investigation into this approach is reported by Drisko (40). Eighteen different additives were evaluated in this study including vinyl acetate polymers, vinyl-acrylic copolymers, acrylic polymers, epoxyesters, dying oils, neoprene, ground tire tread rubber and portland cement. Most of the additives produced significant improvements in abrasion resistance, cohesive strength, and cracking resistance, but the need for full field testing was indicated. The additive cost was modest, increasing the slurry seal application cost from 3 to 7 percent.

Crack filling with bitumen or bitumen-aggregate mixtures prior to placing chip seals or overlays on an old flexible pavement is a time-honored practice and is a necessary preparation on badly cracked roads if subsequent reflection cracking is to be reduced. Thus this kind of simple crack repair can hardly be considered to be innovative. Nevertheless, innovative rehabilitation techniques directed primarily to injection of restorative materials into pavement cracks can be suggested. One example is the use of a combination of reclaimed rubber and asphalt, for which cost and performance benefits have been claimed (41, 42). Another is a combination of synthetic rubber and Trinidad Lake Asphalt which has been tried in England (43). A thorough study of five hot-poured rubberized asphalt sealers, three crack sidewall tackcoats, a paper rope bottom bond breaker, and 40 different application procedures was reported by Wolters (44). This study indicated generally acceptable performance with all of the sealers evaluated, but that overfilling routed and cleaned cracks with no bond breaker or tackcoat gave the best results. It was recommended that such crack sealing not be attempted when crack failure percentage exceeds about 30 percent. Still another crack-filling technique is the application of Sulfur-Asphalt-Sand mixtures (45).

In summary, flexible pavement rehabilitation techniques based on direct alteration of the surface materials appear to be attractive because they are simple to apply and can be inexpensive. However, the

effectiveness of many of the proposed approaches has not been demonstrated fully and convincingly. Judicious consideration of the use of such techniques is necessary.

4.4 Surface Rejuvenation Using Physical Processes

Rehabilitation techniques included in this classification are based on indirect application of energy to the surface layer of a flexible pavement to provide a source of heat and/or to promote chemical changes in the paving materials. Indirect energy application implies energy conversion within the materials. Indications are that these methods are more effective and efficient than the direct transfer of heat as exemplified in the heater-planer technique.

Possible means for indirect energy application include irradiation by electromagnetic or sonic waves or the passage of an electric current through the materials. Two techniques which have been evaluated in this investigation involve microwave irradiation and actinic light irradiation. The other possible approaches to indirect energy application were not evaluated because no evidence could be found that they are practical for flexible pavement rehabilitation.

The feasibility of using microwave irradiation in pavement maintenance and rehabilitation has been shown by Sawyer, et al (46) and Jeppson (47), (48), (49), (50). Full scale equipment is in the development stage, using 950 and 2450 mHz microwaves. Such equipment can heat bituminous concrete layers up to 3½ in. (9 cm) thick in contrast to about 3/4 in. (2 cm) thick for direct heating methods. Of course, after the surface layer is heated by microwaves, one of the mechanical surface manipulation methods (using a planer, miller, or roller) discussed previously must be used to complete the job. However, the combination (of microwave heating and mechanical manipulation) appears to be quite effective. For example, a cost estimate for a large heater-scarifier operation using a microwave heat source is 0.22 $/yd^2 (0.26 $/m^2) for a 3/4 in. (2 cm) layer. Estimated energy requirements for microwave pavement heating range from 8,300 Btu/yd^2 (10.6 x 10^6 J/m^2) to 16,600 Btu/yd^2 (20.9 x 10^6 J/m^2) for a 1 in. (2.5 cm) layer compared with 18,200 Btu/yd^2 (23.0 x 10^6 J/m^2) to 20,000 Btu/yd^2 (25.2 x 10^6 J/m^2) for processing a 3/4 in. thickness in heater-planer operation (see Table 4-1).

Application of microwaves in pavement rehabilitation is in the early stages of development. Accordingly, in making a utility decision analysis of this technique, allowance has been made for development time and costs. Also, at present the cost of the total system is quite high (about $665,000) for early production prototypes, but this cost should decrease significantly with production. Note that even when this high capital cost is written off over a reasonable period (5000 hrs), an acceptable application cost of 0.22 $/yd^2 (0.26 $/m^2) was obtained, and a future downward trend of this cost can be expected.

Special safety problems exist when microwave energy is used (51), but these can be handled by proper design, operation, and maintenance of the equipment. Unlike ionizing radiation (gamma and x-rays), exposure to microwaves has no cumulative effect other than unrecovered tissue damage caused by continuous high-level exposure. A special problem is the case of a passing motorist with a heart pacemaker.

Exposure of bitumens to actinic light tends to accelerate the oxidation and polymerization reactions which increase binder viscosity and thus stiffen the pavement. This process can have beneficial results such as increase in pavement fatigue life which may be outweighed by harmful effects as previously discussed. However, for some types of pavement distress, where use of too much binder or one with too low a viscosity binder results in instability and/or tendency to bleed, treatment by irradiation with actinic light could have net positive results.

The physical basis for this technique has been demonstrated quantitatively. For example, Traxler (57) reported experiments where 10 mm films of asphalt cements were exposed in air to 0.36 m (833 THZ) ultraviolet radiation at 95°F (35°C) and 1000 mW/cm^2 for 18 hours. After such exposure 7 - to 90 - fold increases in viscosity were determined. Under the same conditions without irradiation, relatively small viscosity increases were observed with the same materials.

Equipment specifically designed for actinic light irradiation of flexible pavements does not appear to have been developed yet. Thus neither cost nor energy consumption data are available. However, related equipment for exposing materials to ultraviolet light is available commercially (e.g. germicidal lamps, mineral identification equipment and the

like) which presumably could be adapted for use on the road. The following points should be considered in such an approach:

1. Application of actinic light irradiation is limited to treatment of a few cases of pavement surface distress where binder hardening would yield a net beneficial result.
2. The possibility of acceleration of pavement ageing after such treatment must be evaluated.
3. The technique will be most successful with bitumens that tend to be light-sensitive (Ref. 52 indicates considerable variation among asphalts in this respect).
4. The layer thickness over which actinic light irradiation is effective is limited; probably 1 mm or less.

4.5 Surface Layer Reconstruction

Rehabilitation techniques in this category include in-place reworking of the top layer (2 in. or 5 cm thickness or less) or its partial replacement. These are less drastic pavement treatments than complete removal and replacement of the surface course which can be as costly as overlay procedures and therefore not considered in this study. Costs of techniques involving in-place surface reworking or partial replacement usually will be higher than for application of the more superficial treatments considered in the previous discussion. However, the restorative process is more thorough and a longer subsequent pavement service life can be expected.

In general, surface rework involves some break-up and in-place mixing of the surface materials with additives, followed by lay-down and re-compaction. Heater-scarifiers are most commonly employed to break up the surface layer, which is followed by the other operations. Some heater-scarifier units (offered by Cutler and Jim Jackson Contractors) combine the heating, scarifying, additive injection, mixing, and mix screeding operations. Sometimes a sprinkle treatment of aggregates is used before recompaction to improve skid resistance. A summary of heater-scarifier equipment, costs, and energy requirements is given in Table 4-5. Surface reworking techniques could be improved by modifications of the break-up operation to reduce costs, energy requirements and atmospheric pollution. Possibilities include use of hydraulic or ultrasonic impactors to conduct

Table 4-5. Summary - Comparison of Heater - Scarifier Equipment.

Manufacturer [References]	Equipment Features	Costs			Fuel			Comments
		$/yd^2 ($/m^2)	Includes	Type	Quantity gal/yd^2 dm^3/m^2	Btu/yd^2 (J/m^2)	Includes	
Asphalt Equipment, Inc. (32) (54)	heats and scarifies	0.15-0.28 (0.18-0.33)	equipment rental, fuel and labor to heat and scarify	propane				fire horizontally 3/4 in. (2 cm) scarification
Cutler "Metro" Repaver (55) (56) (57)	heats, scarifies and can add new mix and/or additive in one operation	0.60 (0.72)	equipment rental,	propane	0.09 (0.41)	8,200 (10.3×10^6)	heating	indirect heating 1,000 yd^2 (836 m^2) per hr. 3/4 in. (2 cm) scarification
				diesel	0.015 (0.068)	2,100 (2.6×10^6)	power unit	
Cutler "Jumbo" Repaver (55) (56) (57)	heats, scarifies and can add new mix and/or additive in one operation			propane				indirect heating 3/4 in. (2 cm) scarification
				diesel				
Jim (58) Jackson Heater Scarifier (4)	heats, scarifies and relays scarified mix			propane	0.16 (0.72)	14,400 (18.2×10^6)	heating	3/4 in. (2 cm) scarification
Payne (58) Heater Scarifier (60)	heats and scarifies	0.18 (0.22)	heat and scarify	propane	0.09 (0.41)	8,500 (10.7×10^6)	heating	1,000 yd^2 (836 m^2) per day 3/4 in. (2 cm) scarification
				diesel	0.06 (0.27)	8,000 (10.1×10^6)	power unit	

Table from Epps (7)

the break-up at ambient temperatures.

Much of the distress observed in flexible pavements is localized in the wheel path. Distress problems include rutting, shoving, bleeding, and pot-holes. Accordingly, when most of the remainder of the surface is in good condition, replacement of only the material in the wheel path is worth considering as a rehabilitation technique. Implementation first requires selective removal of surface layer material in the wheel path. This can be accomplished, for example, by one of the traveling road milling machines already discussed. The removed material could be rejuvenated and replaced or, since only a portion (20 to 40 percent) needs to be rehabilitated, the old material could be replaced with higher cost, high performance, bituminous mixes or even by special materials such as sulfur-asphalt-aggregate mixes (62), (63), (64), (65), (66), (67).

Literature specifically documenting field experience with a rehabilitation technique involving selective removal and replacement of surface material in the road wheel paths has not been found. However, this technique combines other proven methods and reasonably reliable estimates of feasibility, life and cost can be made.

The propagation of reflection cracks through bituminous pavement surface courses has been a persistent problem for many years. Thus there is a pressing need for effective surface rehabilitation techniques to help solve this problem. Many studies have been made with most of the work being empirical in nature (68). In flexible pavements, reflection cracks can propagate through the surface layer as a result of thermal stresses, expansion and contraction of untreated base materials, and shrinkage of cement-treated base materials. Reflection of alligator cracking resulting from fatigue failure also occurs, but in such instances the problem should be solved by modification of the base courses rather than by application of surface rehabilitation techniques.

Thick overlays sometimes can defer, but not prevent, reflection cracking. Sherman (68) has reviewed reports on nine different methods which have been proposed for preventing or reducing reflection cracking. Most of these have not been very effective. However, one of the most promising approaches is to interpose a stress-relieving layer between the cracked existing bituminous surface and a seal coat or thin overlay. As indicated

by McCullough (69), this approach should be distinguished from internal reinforcement of an overlay, which may employ the same kind of materials.

Early attempts at interposing such a stress relieving layer using welded wire fabric, expanded metal mesh, wax paper, and aluminum foil were only partially successful (68). Better results appear to have been obtained recently with the use of less rigid layers including a non-woven random mixture of polypropylene and nylon covered polypropylene (70), non-woven polypropylene fabric (71), non-woven high tenacity polyester fabric (72), and a woven glass fiber fabric (73). These materials can be placed over the entire road surface or only over transverse cracks. Field results have varied widely, with performance variations associated with the pavement type, fabric placement techniques, characteristics of the fabrics and other materials used, and environmental conditions in service. Also, in some field studies, an important secondary benefit has been observed: most of these fabrics, as installed, act as moisture barriers, effectively sealing the base course from water intrusion.

Insufficient time has elapsed to fully determine the long-time effects of application of stress-relieving fabric mats, but significant retardation of the propagation of reflection cracks appears to have been demonstrated (74), (75), (76). Costs for implementation of this technique are not excessive; installed cost of most fabrics is the order of $\$1.00/yd^2$ ($\$1.20/m^2$). A number of additional laboratory studies and field trials are now in progress.

A different approach to the construction of stress relieving interfaces is represented by the technique reported by Gallaway (77). In this method the membrane was formed in-situ by a slurry seal machine which lays down a mixture of asphalt emulsion, ground vulcanized scrap rubber (1/8 in. (3 mm) maximum particle size) and sand. The finished interface layer had a composition of 30 volume percent asphalt, 33 percent rubber, and 37 percent sand. A standard chip seal was applied over this layer to form the riding surface. Test data indicated that a ¼ in. (6 mm) layer of this interface material would accomodate a crack extension of approximately 0.20 in. (5 mm) before surface course cracking would occur, compared with approximately 0.038 in. without the membrane. Field trials of this approach are reported by James and Morris and McDonald (79). The costs for

this technique should be somewhat less than for application of conventional slurry seals since about 1/3 of the material used is a solid waste product.

It cannot be expected that the lay-down of a stress relieving mat will constitute a complete flexible pavement surface rehabilitation job. However, the current approach to this technique is innovative, and when combined with the application of a chip seal or thin overlay is potentially more cost effective than standard overlays.

Confidence in the results of application of the stress-relieving concept for solving the reflection cracking problem in bituminous pavements would be enhanced if rational design procedures were available. The analytical problems are difficult, but a beginning has been made. Pretorius and Monismith (80) developed an incremental axisymmetric finite element method for predicting stresses causing pavement cracking due to the shrinkage strains in soil-cement bases. The time dependent (viscoelastic) behavior of all layer materials was used as input in this program. Specific solutions for a layered system containing a stress relieving membrane were not given in the paper, but it appears that the method could be extended to include such cases.

Ramsamooj (81) addressed the application of the principles of fracture mechanics to the problem of initiation and growth of reflection cracks in bituminous overlays, under the combined influence of vehicular loading and temperature changes. A theory of delayed fracture in linearly viscoelastic materials was used in formulating the solution.

Chang, Lytton and Carpenter (83) developed a numerical linear viscoelastic stress analysis for predicting thermal stresses in an overlay and the old bituminous surface. The results of their study indicated that there are three stages of crack growth in an overlay, each requiring a different layered arrangement of material properties to retard crack growth.

Further analytical work is necessary to extend these studies in order to develop a fully predictive technique for crack retardation in a pavement structure containing a stress relieving layer.

4.6 Utility of Potential Surface Rehabilitation Techniques

Surface rehabilitation techniques for flexible pavements considered in this study are listed in Table 4-6. Those not considered to offer any potential advantages over standard overlays and seal coats were

Table 4-6. Surface Rehabilitation Techniques to Solve Flexible Pavement Distress Problems

SURFACE REHABILITATION TECHNIQUE / PAVEMENT DISTRESS PROBLEM	General Deterioration of Surface[a]	Surface Loss[b]	Distortion of Road Profile or Cross-Section[c]	Surface Cracking[d]	Bleeding, Flushing[e]	Holes in Surface[f]
A. Potential Techniques						
1. Mechanical Surface Manipulation						
a. Heater-Planing	✓	✓	✓			
b. Cold Planing	✓	✓	✓			
c. Hot-Milling	✓	✓	✓			
d. Cold-Milling	✓	✓	✓			
e. Heavy Rolling	✓	✓	✓	✓		
2. Surface Rejuvenation by Material (Chemical) Treatment						
a. Rejuvenating Agents	✓					
b. Special Slurry Seals	✓			✓		
c. Crack Sealing				✓		
3. Surface Rejuventation by Physical Methods						
a. Microwave Sealing	✓			✓		
b. UV Irradiation					✓	
4. Surface Layer Reconstruction						
a. Rework Surface	✓	✓	✓	✓	✓	✓
b. Replace Wheel Path			✓		✓	✓
c. Stress-Relieving Interface				✓		
B. Present Techniques						
1. Overlay	✓	✓	✓	✓	✓	✓
2. Chip Seal	✓	✓	✓	✓	✓	✓

Table 4-6. Surface Rehabilitation Techniques to Solve Flexible Pavement Distress Problems (Con't)

Notes:

a. Surface Deterioration
   - Weathering-Hardening
   - Streaking

b. Surface Loss
   - Abrasion and Polishing
   - Raveling

c. Distortion of Profile or Cross-Section
   - Shoving
   - Rutting
   - Corrugation
   - Depression
   - Bump

d. Surface Cracking
   - Alligator
   - Longitudinal
   - Tranverse
   - Edge
   - Block
   - Slippage

e. Holes in Surface
   - Potholes
   - Indentations

discarded during a preliminary screening process (see Appendix B) and are not included. In the tabulation, the techniques evaluated have been grouped into categories consistent with the classification employed in the foregoing discussion, and their applicability to certain types of pavement distress indicated.

As noted previously, none of these techniques should be used where the pavement distress is the result of structural inadequacies of the sublayers (see chapter 3). On the other hand, since most of the techniques are relatively simple and quick to apply, they are generally useful for urban, suburban, or rural highways. There is a wide range of exposure with the techniques listed, from the "tried-and-true" to those having little or no practice in the United States. However, all represent potentially viable alternates to the use of standard overlays or seal coats.

A utility analysis was made of each of the rehabilitation techniques listed in Table 4-6, using the UDAREM program described in Chapter 2. The basis for program input for each of these techniques is indicated in Table 4-7. In most instances, specific input values (e.g., construction costs and energy use) were indicated in previous sections of this discussion. Every effort was made to obtain accurate and realistic data for this input, but, as might be expected, there is a range of reliability of such information consistent with the extent of experience with the techniques.

4.7 Selection of Techniques for Further Study

Utility values generated by the UDAREM program for flexible pavement surface rehabilitation techniques are summarized in Table 4-8. For comparison utility values for standard hot-mix bituminous concrete overlays are also given in this table. For two of the rehabilitation techniques evaluated, the analysis considered their application in Rural, Suburban, and Urban locations. The variation indicated in utility values among locations appears to be much smaller than the variation among the different techniques evaluated. Accordingly, the remainder of the techniques were evaluated for suburban locations only.

In general, the usefulness of a particular surface rehabilitation technique is limited to correcting only a few kinds of pavement distress

Table 4-7. Flexible Pavement Surface Rehabilitation Techniques - Basis for Estimating Decision Criteria Values in Utility Analysis

REHABILITATION TECHNIQUE	Surface Dressing Required?	Development Cost and Time	Capital Investment	Construction Cost	Serviceability Index-Time Relation	User Unit Costs	Maintenance Costs	Pavement Life	SN-Time Relation	Pavement Rating Score-Time Relation	Construction Time	Reworkability	Rehabilitation Energy	Rehabilitation Materials Energy	Impact
Heater-Planing	A	A	A	A	A	U	A	A	A	U	A	B	A	A	A
Cold Planing	A	B	C	B	A	U	A	A	A	U	B	B	A	A	B
Hot Milling	A	B	B	A	A	U	A	A	A	U	A	B	A	A	B
Cold Milling	A	B	B	A	A	U	A	A	A	U	A	B	A	A	B
Heavy Rolling	A	B	A	B	A	U	A	A	A	U	B	B	B	A	B
Rejuvenating Agents	C	C	A	A	C	U	C	A	C	U	A	A	B	B	B
Special Slurry Seals	C	C	B	B	C	U	A	A	A	U	A	A	B	B	B
Crack Sealing	A	A	A	B	A	U	A	A	A	U	A	B	B	B	B
Microwave Sealing	A	B	C	A	A	U	A	A	A	U	B	B	A	A	B
UV Irradiaiton	C	D	C	B	C	U	A	A	C	U	A	D	B	B	B
Rework Surface	C	B	B	B	A	U	A	A	A	U	A	A	A	A	A
Replace Wheel Path	C	B	B	B	B	U	B	A	A	U	A	C	A	A	B
Stress Reueving Interface	A	A	A	B	B	U	A	A	A	U	A	B	B	B	B

Table 4-7. Flexible Pavement Surface Rehabilitation Techniques--Basis for Estimating Decision Criteria Values in Utility Analysis (Continued)

KEY TO SYMBOLS USED

Surface Dressing Requirement

A - Required as follows:
  Optimistic - chip seal
  Most probable - 1 in. overlay
  Pessimistic - 2 in. overlay
B - Required, as specified
C - Not required

Development Cost and Time

A - Ready for immediate application
B - Developed; field trials and feasibility demonstration required
C - Significant modification of existing equipment or materials required plus field trials
D - Equipment research and development required
E - Materials research and development required

Capital Investment

A - All equipment available; in-house or rental
B - Most equipment available; some special items required
C - Special equipment required, not generally available

Construction Cost

A - Based on data presented in discussion
B - Estimated from related construction costs
C - Manufacturers or other estimates

Serviceability-Time Relation

A - Normal flexible pavement service experience
B - Better than A
C - Poorer than A

User Unit Costs

U - Users manual data

Maintenance Costs

A - Pavement restored to normal maintenance expectancy
B - Lower costs than A
C - Higher costs than A

Expected Pavement Life

A - Based on time for pavement to reach serviceability - 2.5
B - Based on time for other pavement performance criteria to reach limit value

Table 4-7. Flexible Pavement Surface Rehabilitation Techniques--Basis for Estimating Decision Criteria Values in Utility Analysis (Continued)

SN - Time Relation

A - Normal S-N time relation indicated in users manual
B - Better than A
C - Poorer than A

Pavement Rating Score - Time Relation

U - Users manual data

Construction Time

A - Based on experience with application of the technique in a given traffic situation (Urban, Suburban, Rural); information presented in discussion
B - Estimate based on time required for related construction activities

Expected Reworkability

A - Same technique can be reapplied indefinitely
B - Reworkable, be number of reapplications are expected to be limited
C - Reworkability is questionable
D - Reapplication of technique is impossible or very unlikely

Rehabilitation Application Energy

A - Based on data given in discussion
B - Estimated from data available on related construction techniques
C - Manufacturers or other estimates

Rehabilitation Materials Energy

A - Based on data given in discussion, plus energy in surface dressing materials where required
B - Estimated from data available on related construction techniques
C - Manufacturers or other estimates

Impact Items

A - Based on current experience with technique, usually indicated in discussion
B - Estimated by general consideration of technique equipment and materials characteristics

Table 4-8. Summary of Utility Decision Analysis Values For Flexible Pavement Surface Rehabilitation Techniques

TECHNIQUE	UTILITY VALUES AT HIGHWAY LOCATION[a]		
	RURAL	SUBURBAN	URBAN
A. Potential Techniques			
Heater-Planing	0.708	0.705	0.692
Cold Planing		0.646	
Hot Milling	0.685	0.689	0.690
Cold Milling		0.705	
Heavy Rolling		0.716	
Rejuvenating Agents		0.799	
Special Slurry Seals		0.793	
Crack Sealing		0.732	
Microwave Sealing		0.654	
UV Irradiation		0.674	
Rework Surface		0.785	
Replace Wheel Path		0.756	
Stress Relieving Interface		0.731	
B. Current Techniques			
Overlays			
1 in. (2.5 cm)		0.705	
2 in. (5 cm)		0.737	
4 in. (10 cm)		0.710	
6 in. (15 cm)		0.696	

[a]See Appendix J for det ils of utility values.

problems, as indicated by the columns checked in Table 4-6. Thus, choice of the "best" surface rehabilitation techniques was made in each instance among those applicable for each of the several kinds of pavement distress problems.

A comparison of the utility values for the surface rehabilitation techniques applicable for correcting bituminous pavement surface loss and deterioration are shown graphically in Figure 4-1. Only three of the techniques evaluated in this category gave utility values significantly higher than those for standard overlays.

Application of Rejuvenating Agents and Special Slurry Seals appear to be the best choices for correction of surface loss and deterioration. However, in the manner these techniques are currently employed, certain problems exist with their application, as indicated in previous discussion. Further study of the application of these techniques should therefore include the development of improved materials and methods.

Note that the pavement planing and milling techniques do not show up well in this comparison, partly as a result of their relatively high cost and energy demands. However, in special applications, such as those where the road surface elevation must be reduced, these techniques might be preferred over others.

The most innovative of these techniques, Microwave Sealing and Cold Planing, have the lowest utility values. These methods appear to have considerable promise, but were penalized in the analysis by the high development cost and time and capital investment required. Accordingly, the Microwave Sealing technique was selected for special consideration.

A similar comparison of utility values for surface rehabilitation techniques suitable for correcting distortion of the road profile or cross-section is shown in Figure 4-2. In this category, only two of the techniques, Rework and Surface and Replace Wheel Path, have utility numbers higher than those for standard overlays. The Rework Surface technique has a special appeal because the highway surface materials are completely recycled. Also in this category, the various surface milling and planing techniques were not selected but could be useful

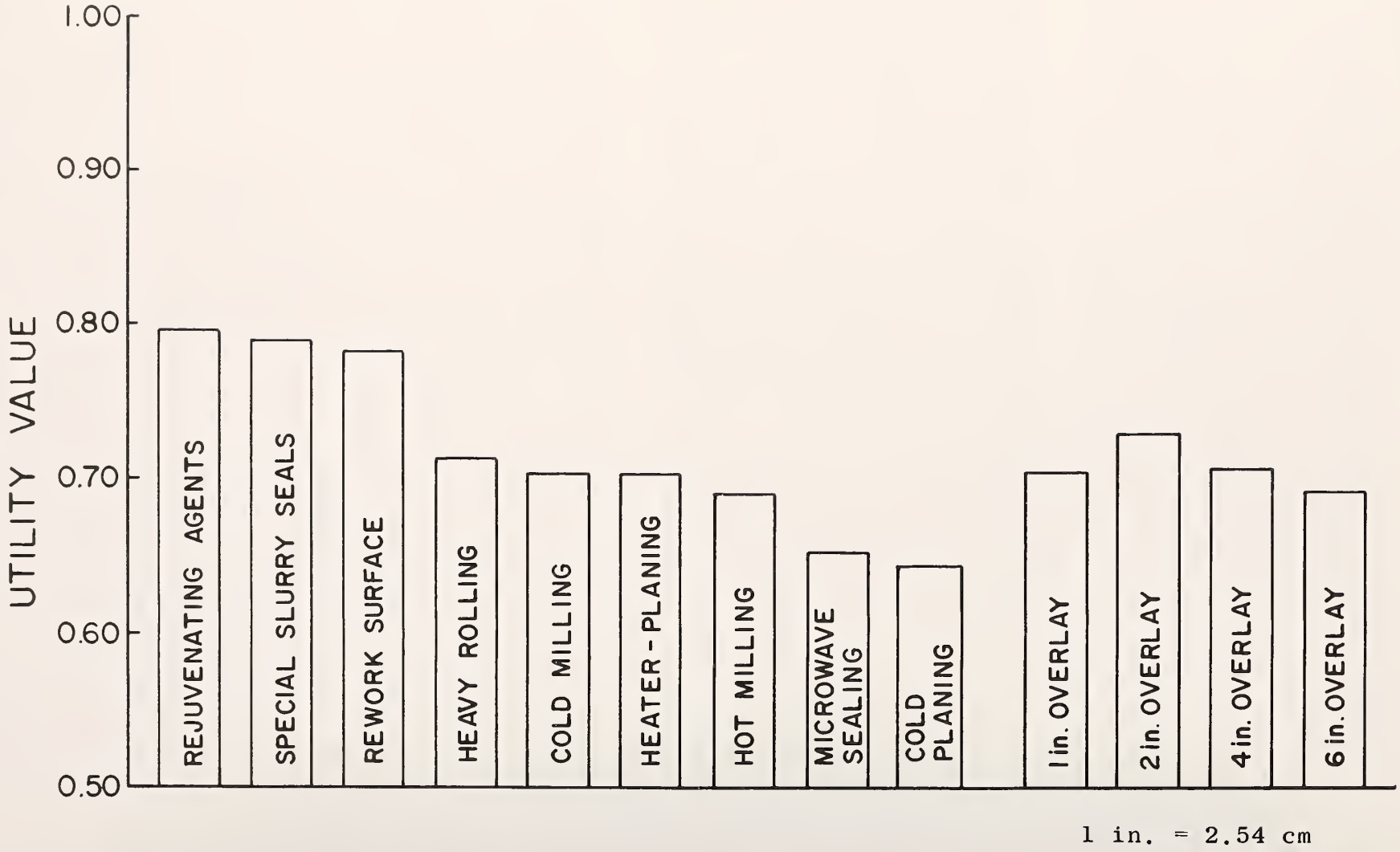


Figure 4-1. Comparison of Rehabilitation Techniques for Correction of Flexible Pavement Surface Loss and Deterioration.

Figure 4-2. Comparison of Rehabilitation Techniques for Correction of Distortion of Flexible Pavement Profile or Cross-Section

in special situations.

A comparison of utility values for the six surface rehabilitation techniques useful for correcting flexible pavement surface cracking is given in Figure 4-3. In this category the first and second choices appear to be Special Slurry Seals and Rework Surface, both of which are also suitable for other flexible pavement distress problems. A special choice which has a slightly lower utility value is the application of Stress Relieving Liners. This approach deserves special consideration as a result of its potential for alleviating reflection crack problems.

Migration of the bituminous binder to the surface can be a particularly troublesome problem with flexible pavements. Not only does such bleeding result in a serious accident hazard, but correction of the problem is not easy to achieve. The two best techniques proposed to overcome this problem, as indicated by the utility values listed in Table 4-9 are Rework Surface and Replace Wheel Path. The utility values for both of these techniques are higher than those for standard overlays. They are also indicated for correction of surface holes and indentations, Table 4-10. The use of the UV Irradiation is innovative, but in view of the low utility value listed and its limited application this technique is not believed to be worthy of further study.

The first, second, and special choices of surface rehabilitation techniques for the various flexible pavement distress problems considered in this study are summarized in Table 4-11. Six (out of thirteen) techniques are indicated for further study, evaluation, and development. The extent and nature of this work has already been indicated in previous sections of this discussion.

Finally, the range of applicability of each of the six techniques selected will be of interest and is indicated in Table 4-12. Thus if both utility and range of applicability are considered, the "best" among all of the surface rehabilitation techniques evaluated is the Surface Rework approach, which also has other benefits as already discussed.

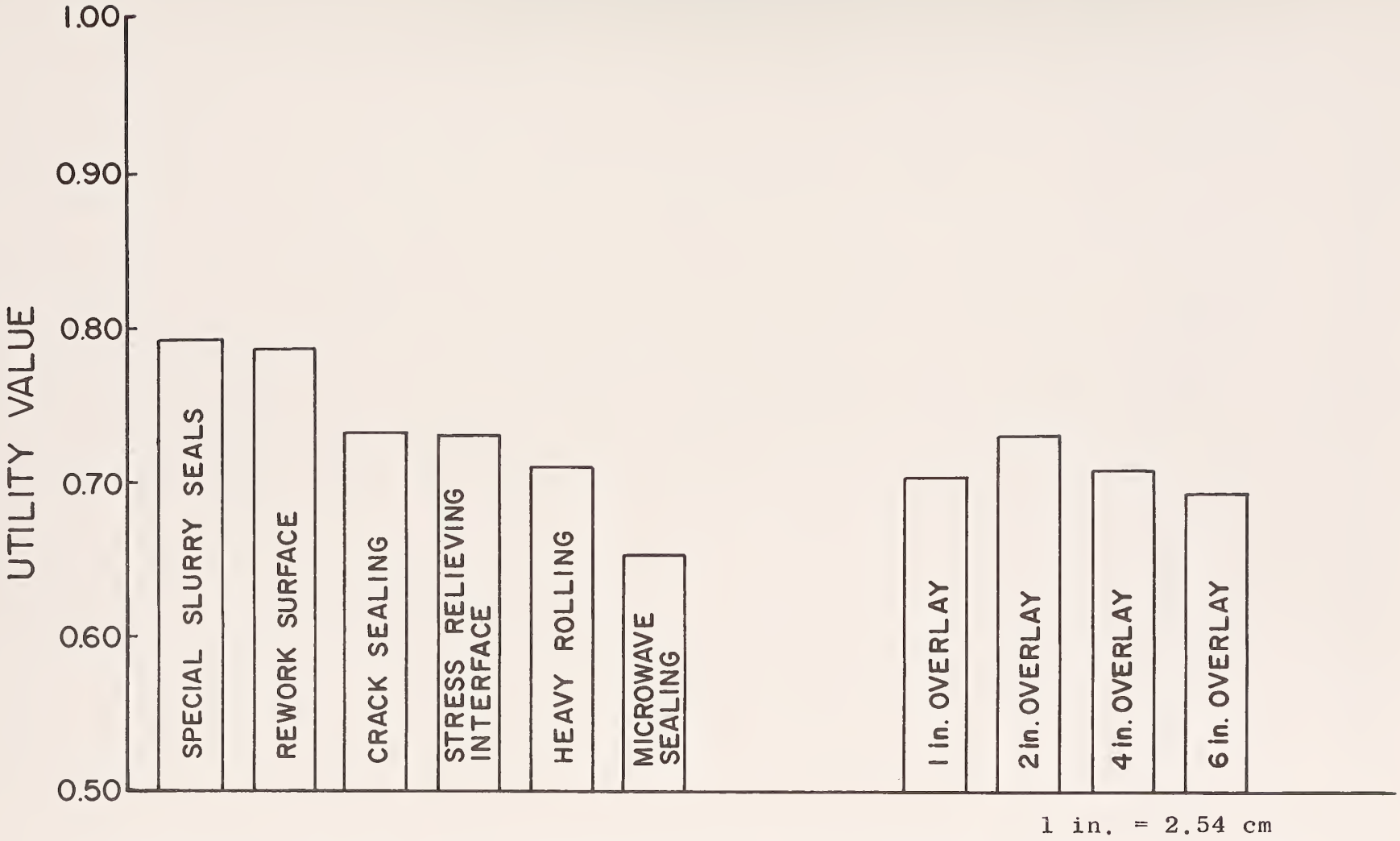


Figure 4-3. Comparison of Rehabilitation Techniques for Correction of Distortion or Flexible Pavement Cracking

Table 4-9. Utility Comparison of Flexible Pavement Surface Rehabilitation Techniques for Correction of Bituminous Binder Bleeding

TECHNIQUE	UTILITY VALUE FOR SUBURBAN LOCATION
A. Potential Techniques	
Rework Surface	0.785
Replace Wheel Path	0.756
UV Irradiation	0.674
B. Current Technique	
2 in. (5 cm) Overlay	0.737

Table 4-10. Utility Comparison of Flexible Pavement Surface Rehabilitation Techniques for Correction of Holes and Indentations

TECHNIQUE	UTILITY VALUE FOR SUBURBAN LOCATION
A. Potential Techniques	
Rework Surface	0.785
Replace Wheel Path	0.756
B. Current Technique	
2 in. (5 cm) Overlay	0.737

Table 4-11. Summary of Flexible Pavement Surface Rehabilitation Technique Selections

PAVEMENT DISTRESS PROBLEM	REHABILITATION TECHNIQUE SELECTION			
	FIRST CHOICE	SECOND CHOICE	SPECIAL CHOICE	JUSTIFICATION
Surface Loss and Deterioration	Rejuvenating Agents	Special Slurry Seals	Microwave Sealing	Promising Development Potential
Distortion of Profile or Cross-Section	Rework Surface	Replace Wheel Path		
Surface Cracking	Special Slurry Seals	Rework Surface	Stress Relieving Interface	Reduction of Reflection Cracking
Asphalt Bleeding	Rework Surface	Replace Wheel Path		
Holes and Indentations	Rework Surface	Replace Wheel Path		

Table 4-12. Range of Applicability of Selected Flexible Pavement Surface Rehabilitation Techniques

Technique	Distress Problem Basis for Selection
Rejuvenating Agents	General Surface Deterioration
Special Slurry Seals	Surface Loss and Deterioration; Surface Cracking
Rework Surface	Distortion of Road Profile or Cross-Section; Surface Cracking; Asphalt Bleeding; Holes and Indentations
Replace Wheel Path	Distortion of Road Profile or Cross-Section; Asphalt Bleeding, Holes and Indentations
Stress Relieving Liner	Surface Cracking (Reflector Cracking)
Microwave Sealing	Surface Loss and Deterioration

5. GEOMETRIC REVISION OF FLEXIBLE PAVEMENTS

5.1 Effect of Geometric Revision

Geometric design of highway pavements is generally dictated by consideration of vehicle dynamics, highway capacity, and safety rather than pavement structure. For example, early studies by the Bureau of Public Roads (1) on two and four lane highways indicated that commercial vehicles remained safely spaced only when lane widths reached 12 ft (3.7 m). As a result of these and other studies, AASHTO (2) recommends 12 ft (3.7 m) lane widths. On multilane divided highways a 12 ft (3.7 m) lane width is justified mainly on the basis of maintaining highway capacity. Even a 1 ft (.3 m) reduction in lane width of a multilane highway can result in a capacity reduction of 10 percent, depending also on lateral clearance (shoulder width) (3).

More recent studies of truck placement on four-lane freeways with 12 ft (3.7 m) lanes, reported by Emery (4), indicated that such vehicles tend to travel to the right (1/2 ft to 3/4 ft) (15 cm to 23 cm) of the centerline of the outside traffic lane. Also, numerous encroachments on the paved outside shoulder were observed in this study. As a result, the author recommended the construction of 13 ft (4 m) traffic lanes, particularly for rigid pavements. Pavement widths greater than 13 ft (4 m) are not desirable because wider lanes tend to encourage reckless drivers to tacitly (and unsafely) create another traffic lane.

Generally, traffic lanes of most Interstate and other first-class highways in the United States are 12 ft (3.7 m) wide, with a few being 13 ft (4 m). Structurally, flexible pavements of such dimensions are not used efficiently because, even with the occasional wandering of heavily loaded vehicles, the standard deviation of the transverse traffic distribution is approximately

1 ft (.3 m) (5). Thus only a relatively small portion of the roadway is subjected repetitively to maximum wheel loads. Such localization of the load results in concentration of two types of pavement distress in the wheel paths:

1. Ruts, which are caused either by surface wear and polish or permanent deformation of the base courses and subgrade.
2. Fatigue failure of the surface course (particularly in full-depth and other thick bituminous concrete pavements).

With this concept in mind, one of the proposed techniques involves displacement of the traffic pattern by lateral movement of the lane markings. This approach could be useful, provided:

1. Pavement distress (rutting and alligator cracking) has not progressed too far. That is, the proposed technique represents a pre-emptive rehabilitation approach.
3. The total pavement width will permit a significant lateral shift in the wheel path locations.

The effect of localization of high stress applications in the wheel path on the performance of a full-depth airfield taxiway pavement was indicated in a paper presented by Witczak (6). These data, shown in Figure 5-1, indicated the fatigue failure (evidenced by cracking) resulting from repeated strain applications in the wheel path. Comparable results can be expected for thick bituminous highway pavements subjected to heavy truck traffic.

An example illustrating the increase in pavement life which might be expected as a result of lateral shift can be given, using data on a

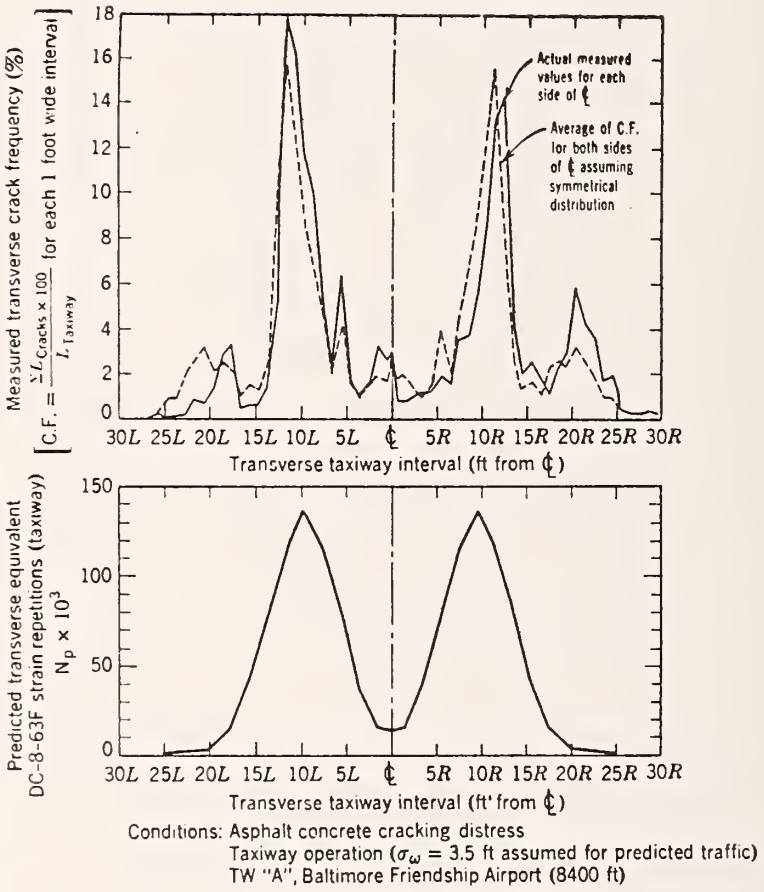

Measured transverse crack frequency (%)
[C.F. = ΣL Cracks × 100 / L Taxiway for each 1 foot wide interval]
Actual measured values for each side of ℄
Average of C.F. for both sides of ℄ assuming symmetrical distribution
30L 25L 20L 15L 10L 5L ℄ 5R 10R 15R 20R 25R 30R
Transverse taxiway interval (ft from ℄)
Predicted transverse equivalent DC-8-63F strain repetitions (taxiway) Np × 10³
30L 25L 20L 15L 10L 5L ℄ 5R 10R 15R 20R 25R 30R
Transverse taxiway interval (ft from ℄)
Conditions: Asphalt concrete cracking distress
Taxiway operation (σω = 3.5 ft assumed for predicted traffic)
TW "A", Baltimore Friendship Airport (8400 ft)

Figure 5-1. Wheel path cracking and strain applications.

1 ft = 0.3 m

From Witczak (6).

full scale test road reported by Hicks and Monismith (7). The pavement structure consisted of 3.1 in. (7.8 cm) of asphaltic concrete on 14.6 in. (45 cm) of compacted aggregate base on a sandy-clay subgrade. Deflections were estimated using a finite element analysis of the structure represented by Figure 5-2. Estimated and measured surface and structural deflections are shown in Figure 5-3.

Using equations given by Ahlvin and Ulery (8), and the parameters for this example, the following strains can be estimated from the deflections:

$$\varepsilon = 550\ \Delta_z \times 10^{-6} \text{ in./in. (cm/cm)}$$

where

ε = radial strain at bottom of surface course

Δ_z = vertical deflection, in. (cm/2.54)

For this type of pavement the fatigue life can be estimated from empirical equations such as those presented by Epps and Monismith (9) and Kallas and Puzinauskas (10). For this example, the following equation was selected:

$$Ni = 2.7 \times 10^{-7}\ (1/\varepsilon)^{3.25} \quad \text{.....(5-1)}$$

To complete the analysis, Miner's hypothesis of cumulative damage was used:

$$\Sigma \frac{n_i}{N_i} = 1 \quad \text{.....(5-2)}$$

where

n_i = number of strain applications at strain level, i.

N_i = number of strain applications to cause failure at strain level, i.

From Figure 5-3, maximum deflections under the wheel path is approximately 0.015 in. (0.24 cm). Thus $\varepsilon = 825 \times 10^{-6}$. This yields $N_i = 2800$ cycles (wheel load repetitions). Since the technique is pre-emptive,

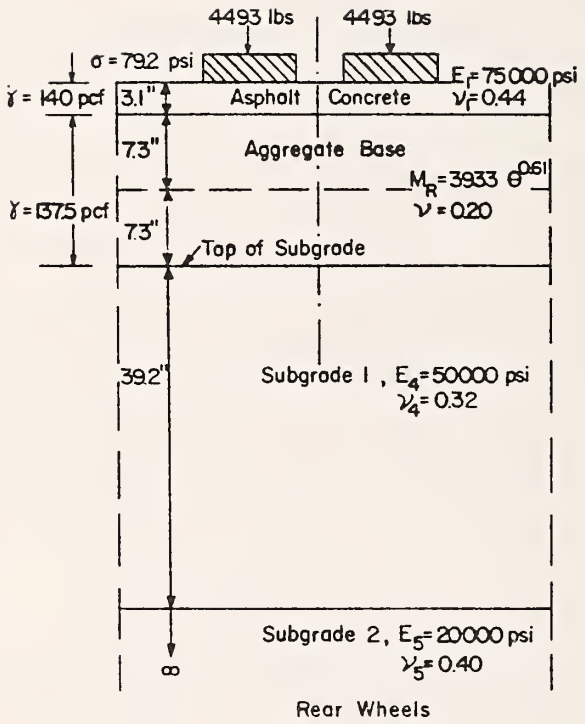


Figure 5-2. Representation of test road for structural analysis.

1 in. = 2.54 cm
1 lb = 0.454 kg
1 psi = 6.9 kPa
1 pcf = 4.9 kg/m^2

From Reference 7.

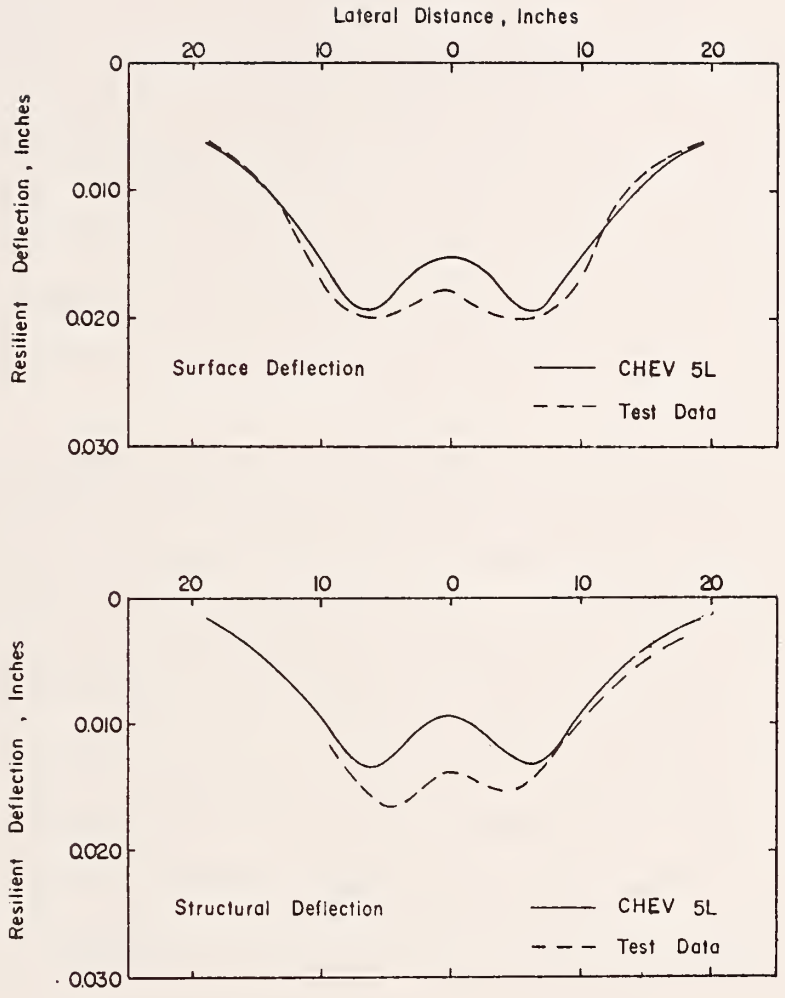


Figure 5-3. Predicted and measured test road deflections.

1 in. = 2.54 cm

From Reference 7.

assume that the wheel paths are shifted at N = 2000. If the shift is 2 ft (0.6 m), less than 0.5% of the fatigue life will have been used up under the new maximum deflection location and subsequent deflections under the original wheel path will approach zero. Thus the total pavement life will increase to 2000 + 2790 = 4790 cycles. The net result is, then, a 70 percent increase in pavement life, limited by fatigue cracking. If the wheel path is moved only 1 ft (0.3 m), about 19% of the fatigue life will have been used up under the new wheel path and the subsequent life is 2270 cycles for a total of 4270 cycles. The net result in this case is a 50 percent increase in pavement life.

Similar considerations would govern the effect of shifting the wheel path locations on rutting caused by permanent sub-layer distortion, although different data would be used. Pavement life limited by rutting caused by wear should be doubled by application of this technique.

This example suggests that, for a four-lane highway, the technique would work best for 13 ft (4 m) lanes, which would allow a lateral traffic shift of 2 ft (0.6 m). Since truck encroachment on the outer shoulder should be minimized, the lane displacement of 1 to 2 ft (0.3 to 0.6 m) should always be to the left, in the direction of traffic movement.

Costs for applying this technique should be minimal, being principally the costs for remarking the traffic lanes. However, the provisions previously mentioned must be carefully observed for this technique to be applied successfully.

Another proposed geometric revision rehabilitation technique requires reversal of the pavement cross slope as illustrated in Figure 5-4. This technique would only be applicable to divided multi-lane highways and would also require some modification of the highway drainage system and some excavation and fill. These operations will obviously add to the

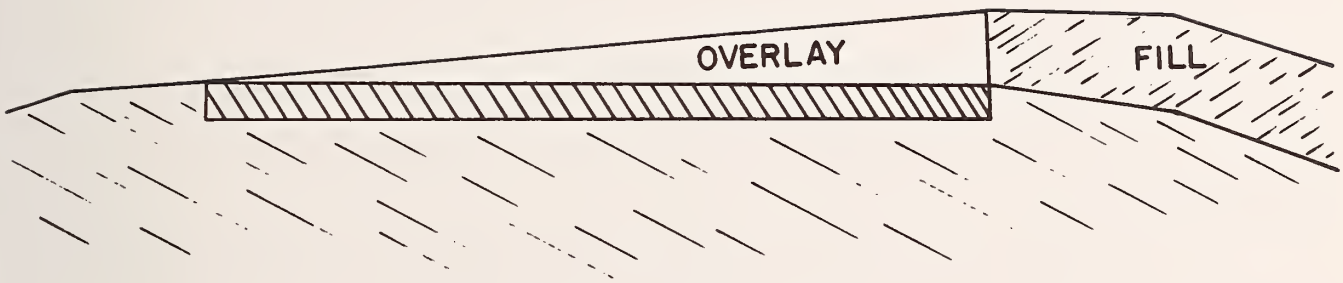


Figure 5-4. Use of overlay to change cross-slope.

cost.

The advantage of this approach over the use of a standard overlay is that the greatest depth of material is laid over the outside traffic lane. This lane is nearly always more distressed than the inner lane. If the cross-slope is 1/8 in./ft (8 cm/m), the outer lane would be covered with a depth of new asphalt concrete varying from 3 in. (8 cm) to 6 in. (15 cm), while the overlay on top of the inner lane would vary from a feather edge to 3 in. (8 cm) thick. Essentially, the paving cost would be about 1/2 that of a standard 6 in. (15 cm) overlay, but should result in about the same performance.

However, some problems must be considered in each application, including:

1. The required drainage revisions and earth moving operations must be practicable.

2. Special provisions must be made on horizontal curves, to provide the proper superelevation. This could be particularly troublesome where spiral transition curves are applied.

3. Paving equipment must be capable of handling the thin overlay of variable thickness for the inner lane.

4. The technique will be advantageous only when there is significantly greater pavement distress in the outer traffic lane.

5.2 Assessment and Selection of Geometric Revision Techniques

A utility analysis was made of the two geometric revision techniques considered in the foregoing discussion, using the UDAREM program described in Chapter 2. The basis for program input is indicated in Table 5-1. Complete details for the program input is given in Appendix I. Based on this utility analysis, the relative values of the two techniques considered, compared with standard overlays, are:

Table 5-1. Geometric Revision Techniques - Basis For Estimating Decision Criteria Values For Utility Analysis.

DECISION CRITERIA	REVISION TECHNIQUE: MOVE LANE MARKINGS	REVERSE DIRECTION OF CROSS - SLOPE
Development Cost and Time	Work required to develop analytical schemes to establish conditions (pavement distress creteria, wheel loadings, pavement width and thickness delineating range of applicability.	No additional development work required.
Capital Investment	No new equipment required	No new equipment required
Construction Costs	Cost estimate for pavement restriping. (11)	Cost for fill, drainage adjustment plus ½ cost of 6 in. overlay (See discussion)
Serviceability Index-Time Relation	Based on extension of service life limited by fatique failure as indicated by example in discussion.	Assume performance will be the same as for 6 in. overlay.
User Unit Cost	See Appendix I	See Appendix I
Maintenance Cost	Pavement will have normal maintenance expectancy	Pavement restored to normal maintenance expectance.
Pavement Life	Estimated time for pavement SI to reach 2.5.	Estimate time for pavement SI to reach 2.5
Level of Pavement Service	Use same data as for 2 in. overlay.	Use same data as for 6 in. over lay.
Construction Time	Time estimate for pavement striping (very short).	3/4 of time to construct 6 in. overlay plus allowance for fill and drainage adjustments.

1 in. = 2.54 cm

Table 5-1. Geometric Revision Techniques - Basis for Estimating Decision Criteria Values For Utility Analysis (Continued)

DECISION CRITERIA	REVISION TECHNIQUE	
	MOVE LANE MARKINGS	REVERSE DIRECTION OF CROSS - SLOPE
Reworkability	Possibility of reapplication of technique very remote	Elevation changes will usually rule out reapplication of technique.
Rehabilitation Energy	Estimate of energy for stripe application.	½ of energy cost for constructing 6 in. overlay plus energy for fill and compaction.
Rehabilitation Materials Energy	Energy to process the quantitity of paint estimated from ref (11)	½ of materials energy for 6 in. overlay
Impact	low impact, high ratings assumed	same as for 6 in. overlay

1 in. = 2.54 cm

Geometric Revision Technique	Utility Value for Suburban Location
Change Lane Markings	0.785
Reverse Direction of Cross-Slope	0.681
1 in. Overlay (2.5 cm)	0.705
2 in. Overlay (5 cm)	0.737
4 in. Overlay (10 cm)	0.710
6 in. Overlay (15 cm)	0.696

Evidently, obtaining more efficient utilization of a flexible pavement structure by traffic redirection through lane marking changes is an attractive method of extending pavement life, when the conditions are such that this approach is feasible. That is, when lane widths, wheel loading, and pavement structural details are favorable. Conversely, cross-slope reversal appears to offer no benefits over the use of standard overlays, probably because excavation and drainage costs more than offset any possible savings in paving cost.

6. REPAIR OR REPLACEMENT OF FAILED JOINTS

6.1 Introduction

Portland cement concrete is a volume unstable material! It starts its "life" as a liquid paste in which are suspended fine and coarse aggregate particles. Through a chemical process called hydration, the cement and water forming the paste combine and harden. Bonds are developed between the particles of cement and between the cement and the aggregate surfaces. A gel-like structure is formed and in real life hydration, moisture is always lost from the system, with the result that the volume of the concrete is always reduced. Thus the concrete is said to have undergone "shrinkage" due to hydration. Early in the life of the concrete there is a race between the gradually increasing strength from the developing gel structure and the gradually increasing stress from restrained hydration shrinkage (shrinkage being restrained by the developing gel structure). The shrinkage almost always wins out in this race with the result that the restrained shrinkage stresses eventually exceed the concrete's tensile strength and a crack is formed. In a concrete pavement, restraint also exists from the friction between the slab and the subbase; if reinforcing steel is present, it too provides a restraining force against shrinkage. C. P. Vetter, in 1933 (1), mathematically defined the magnitude of the shrinkage stress as a function of the shrinkage coefficient of the concrete and the amount of reinforcing steel. Normal concrete, as it hydrates and dries out, will shrink from 500 to 1200 x 10^{-6} in./in. (cm/cm) (2), with an average value of around 900 x 10^{-6} in./in. (cm/cm) (3). The concrete's tensile strain capacity ranges from 100 to 600 x 10^{-6} in./in. (cm/cm) depending upon strength and age, with an average value of around 300 x 10^{-6} in./in. (cm/cm), or less than one-third the shrinkage strain - if fully restrained. The results are as sure as death and taxes - cracks form.

Another, related, volume change occurs. There is a volume change due to temperature changes. Concrete possesses a thermal coefficient of expansion of between 4 and 7 x 10^{-6}/°F (2 and 4 x 10^{-6}/°C) (3). Again, a pavement slab is restrained from moving by the subbase friction with the result that tensile stresses are built up as the temperature drops. Compounding the problem further, there is often a temperature gradient

between the top and bottom of the slab causing a warping stress.

The result of all these restrained volume changes on concrete pavement is the simple fact that concrete of any significant size will always crack. Generally it will crack within the first few days of its life, and subsequently the crack, or cracks, will widen and/or become narrower depending upon temperature changes. The Michigan test road results showed that a joint spacing of around 10 ft (3 m) was necessary to completely prevent transverse slab cracking in jointed pavement (4). The presence of reinforcement in this type of pavement does not reduce the cracking, as both reinforced and non-reinforced pavements will crack approximately the same (4). In a New York study it was found that most of the 61 ft - 10 in. (18.54 m) long jointed reinforced pavement sections developed cracks in the middle third of each section (5). A vivid example of what happens to fully restrained concrete pavements is the crack pattern developed in continuously reinforced concrete pavement (CRCP). Here the combination of reinforcing steel and subbase friction are utilized to induce cracking - closely spaced - in the pavement (6). The resulting cracks are often spaced around five ft (1.5 m) apart and are held tightly closed by the reinforcing steel (6). What do engineers do about this cracking? They sometimes ignore it (with the result that a meandering crack becomes visible and a problem), or they design around it utilizing the cracks in a fully restrained way (CRCP), or they make it look planned by creating joints.

A joint is nothing more than a controlled crack; controlled in the sense that the crack will occur where the engineer wants it to occur. Depending upon the design he may want closely spaced, tight, joints with sufficient aggregate interlock to provide for load transfer across the joint. Or he may prefer joints spaced further apart and be willing to accept the fact that these joints will move more and thus will not generally have sufficient aggregate interlock to provide for local transfer across the joint. The joint allows for slab movement from hydration shrinkage, temperature changes, and temperature differentials between the top and bottom of the slab (termed "curling" or "warping"). Joints also allow for construction to be halted at the end of each day.

Concrete pavements in the United States had their beginnings in Bellefontaine, Ohio, in 1891 (7), in which small-sized slabs were placed (presumably for ease in construction). Then through many years of trial and error joint designs and construction procedures have developed to the point where four types of joints are presently used (Figure 6-1).

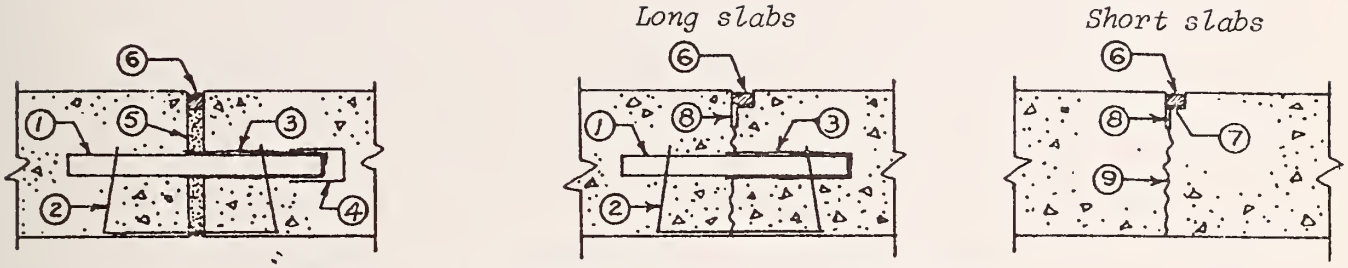
The contraction joint is generally sawed or formed to a depth of T/6 to T/3 thus creating a weakened plane for the formation of a crack as the concrete shrinks (contracts). Load transfer is provided from either aggregate interlock alone or in combination with a load transfer device, of which there are many (7). Longitudinal contraction joints are generally provided between each lane (12 ft or 3.7 m). Spacing of transverse joints utilizing aggregate interlock for load transfer are generally from 15 to 20 ft (4.5 to 6 m) (7), to prevent random transverse cracking from occurring. Such joints do not always function properly! For longer joint spacings, load transfer is provided as the joints open too wide for aggregate interlock. For such longer slabs the pavement is almost always reinforced as the pavement will always crack between the joints (4 , 5), and the reinforcement is necessary to keep the cracks tightly closed. Load transfer, normally dowels, are often misaligned and difficult to keep adequately sealed (5). Thus, the engineer has two less-than-satisfactory choices; either put many close spaced (15 ft or 4.5 m) joints which are maintenance problems, or place fewer joints 30 to 60 ft apart (9 to 18 m) and be plagued with larger joint movements and random slab cracking.

The warping, or hinge, joint is normally a controlled crack to reduce the curling effects from temperature differentials between the slab top and bottom. The joint is held tightly closed with tie bars or distributed steel reinforcement.

The expansion joint is generally a full-depth, formed joint, with load transfer and provision for concrete expansion as well as contraction. Expansion joint spacings vary widely from state to state. Many, if not most, states use expansion joints only at bridge approaches. Some states use expansion joints at approximately 1000 ft (300 m) intervals to relieve expansive forces as the pavement joints fill with deleterious material with age (8).

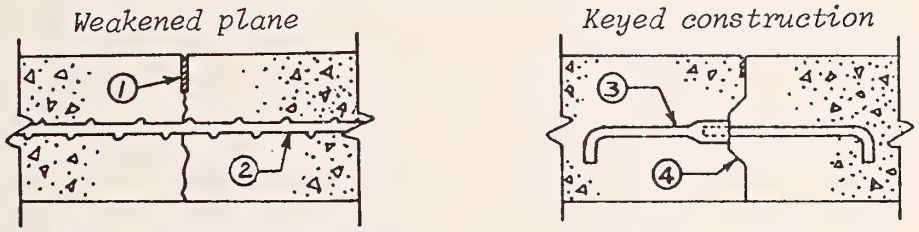
(a) Expansion Joints

(b) Contraction Joints



1. *Dowel*
2. *Basket*
3. *Coating*
4. *Expansion cap*
5. *Filler board*
6. *Seal*
7. *Stop strip*
8. *Groove*
9. *Interlocking aggregate*

(c) Between-Lane Joints



1. *Groove with sealant or insert*
2. *Deformed steel tie*
3. *Threaded insert tie (or bent re-bar)*
4. *Molded key and keyway*

From Reference 8.

Figure 6-1. Joint Types.

The construction joint is a full-depth formed joint constructed whenever concrete placement is halted for any significant period of time. It may be either an expansion or contraction joint, but load transfer devices are always provided.

Joints are sealed with a variety of materials and techniques, ranging from no seal on some contraction and warping joints to expensive preformed sealers for contraction and expansion joints (7). Currently used joint sealants include hot-poured rubberized asphalt, upgraded hot-poured rubberized asphalt, cold poured urethane elastomers, cold-poured polysulfide elastomers, and preformed elastomers (normally neoprene). None of these sealants are completely satisfactory (7).

6.2 Present Joint Design Procedures

Experience and theory have been coupled together to develop several joint design methods. Designs range from a synthesis of accepted practice (8) to sophisticated theoretical design procedures (9, 10). In essence, the major problems to be solved are (a) adequately seal the joint for a prolonged period of time under the destructive effects of pavement movement, traffic, climate, and weather and (b) maintain adequate load transfer for the life of the joint. If the joint is sealed adequately the second problem seems to be minimized.

6.3 Joint Problems

Along with the simple fact that portland cement concrete pavements crack at approximately 15 ft (4.5 m) intervals regardless of what special procedures or techniques are tried (including reducing subbase friction and providing distributed reinforcement), there is the second simple fact that, to-date, none of the miriad of joint designs, sealants, and construction techniques work for very long! As aptly stated by Cook (7):

> At the present time most highway engineers recognize that a large share of the maintenance dollar is spent on joints. Consequently, there is research being conducted at present on pavement movement, joint design and joint sealing. The most widely-used joint in the country today is the contraction joint in the reinforced pavement. This joint is sawed, rather than formed, and is sealed at the top only with preformed compression seals. Joint spacings and subbase type still vary. Highway researchers are not satisfied that this is the optimum joint. The researchers do not believe that they have all the answers, but they do feel that they are at least asking the right questions.

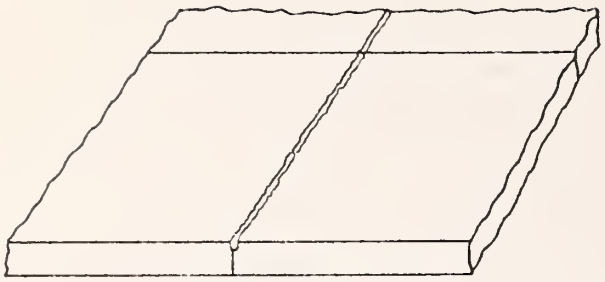
Thus the problem with pavement joints continue to plague highway designers and maintenance engineers. These problems are manifested in a number of ways, which have been categorized as shown in Figure 6-2. The causes of these distress manifestations are (8):

1. Poor slab supports
2. Soil movements
3. Excess water
4. Particle infiltration
5. Faulty load transfer performance
6. Overloads on the pavements
7. Sealant failures
8. Incompatibility of spacing, reservoir, and materials
9. Disintegrated or "rotted" joint faces
10. Improper construction practices.

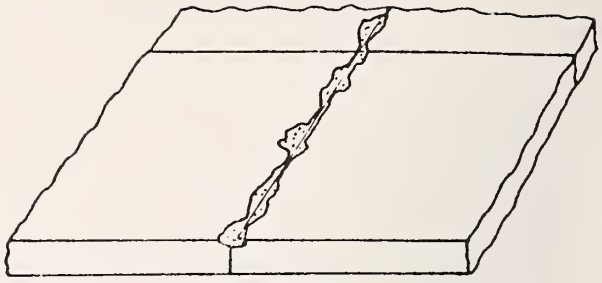
The interrelationship between distress manifestations and possible causes of the distress is shown in Table 6-1. Interaction between causes may accumulate distress and thereby rapidly increase deterioration. Some of these causes can be overcome through improved design; some can be overcome, or at least delayed, through preventative maintenance. Existing problems must be solved through restorative rehabilitation. Currently there are a number of techniques for rehabilitating distressed joints. The excellent NCHRP Synthesis (8) describes many of these techniques. Under preparation is another synthesis (11), which describes in detail both current and some potential new techniques for rehabilitating pavement joints. A third publication presents an excellent state-of-the-art summary of jointed pavement problems and suggested remedies (12). Therefore such current techniques will not be further described here.

6.4 Potential Joint Treatment Techniques

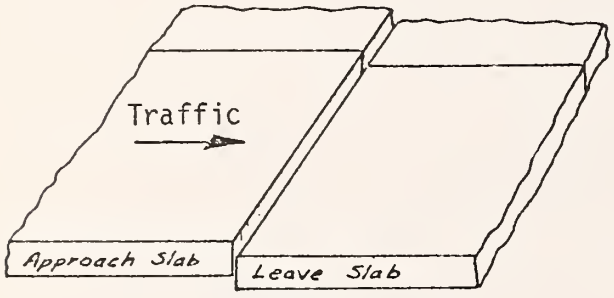
A number of joint repair and/or replacement techniques have been either tried on an experimental basis or proposed for trial. Following is a discussion of each of these techniques which have been judged to have some potential for solving selected problems. (Other techniques, judged as not having any potential for solving the joint problems are discussed in Appendix B).



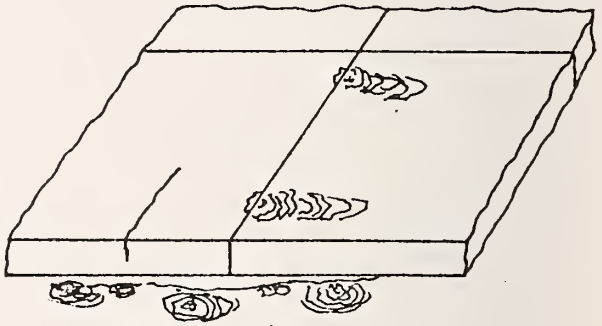
(a) Raveling



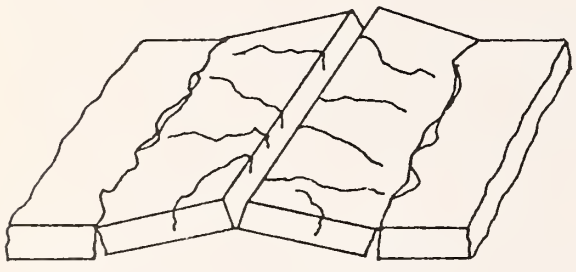
(b) Spalling



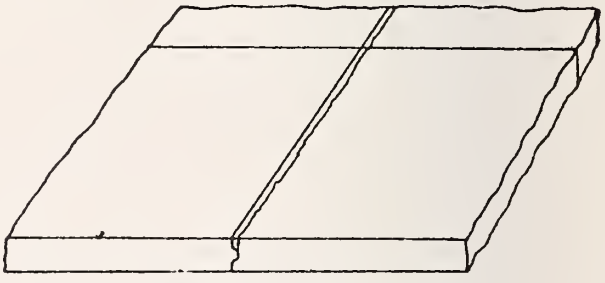
(c) Faulting



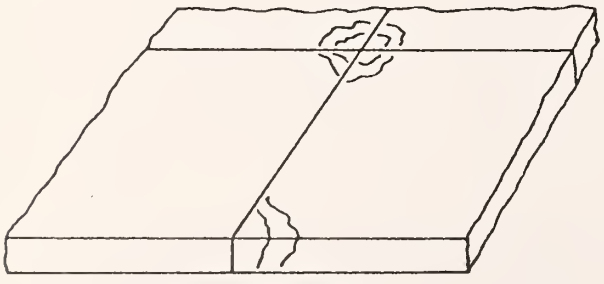
(d) Pumping



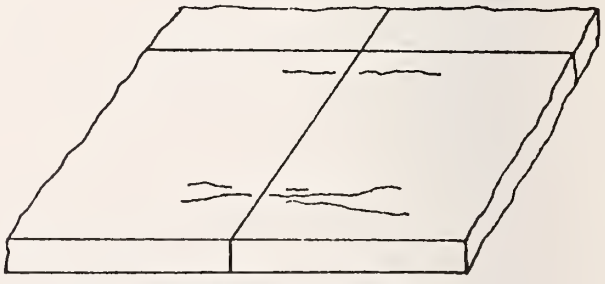
(e) Blowup (buckling)



(f) Keyway Failure



(g) Corner Breaks



(h) Compression Cracks

From Reference 8.

Figure 6-2. Joint Distress Conditions.

Table 6-1. Distress Mainfestations in PCC Joints and Their Possible Causes.

DISTRESS MANIFESTATIONS / POSSIBLE CAUSE(S).	Raveling	Spalling	Faulting	Pumping	Blowup	Keyway Failure	Corner Breaks	Compression cracks
Poor Slab Support			✓			✓	✓	
Soil Movements			✓		✓	✓	✓	
Excess Water			✓	✓			✓	
Particle Infiltration		✓			✓			✓
Faulty Load Transfer Performance		✓	✓			✓	✓	
Overloads on Pavement		✓	✓	✓		✓	✓	
Sealant Failures		✓	✓	✓	✓	✓	✓	✓
Incompatibility of Spacing, Resevoir, and Materials		✓	✓	✓	✓	✓	✓	✓
Disintegrated or Rotted Joint Faces		✓			✓	✓	✓	✓
Improper Construction Practices	✓	✓	✓	✓	✓	✓	✓	✓

6.4.1 Joint Restoration with Improved Joint Sealants. The main idea here is to (a) prevent intrusion of surface water into the pavement sublayers, and (b) prevent intrusion of foreign matter into the joint. Pavement joint sealants have evolved from the hot-poured asphalts, which have relatively short lives, to sophisticated preformed elastomers. Time-and-time again surface water intrusion through a joint or crack accelerates distress mechanisms and promotes early pavement failure. All too often the first-cost economics dictate the type of joint sealant to use. A comparison of joint sealant costs (materials only) for a 24 ft (7 m) length, in 1974, are given in Table 6-2. Experimental results on the performance of these sealants indicate the neoprene preformed sealant was best (13).

Even though the data and theory clearly show that joint shape is one of the most important factors to joint sealant performance (9), many agencies make the joint as narrow as possible to reduce initial sealing costs. The results are always poor, as experience indicates (13). In another study (14) the cost (1974) of making a second cut in an existing pavement to widen the top of a joint and then sealing the joint with improved materials were:

Preformed seal cost per lineal ft (0.3 m)

Second saw cut, ¼ x 1 1/8 in. (0.63 x 3.2 cm)	$0.14
Materials	0.28
Installation	0.15
Total, per ft (0.3 m)	$0.57

PVC hot-poured cost per lineal ft (0.3 m)

Second saw cut, 3/8 x 1¼ in. (0.95 x 3.7 cm)	$0.16
Material	0.076
Installation	0.07
Total, per ft (0.3 m)	$0.306

This compares to a cost per ft (0.3 m) for hot-poured asphalt (less the saw cut) of $0.07 (14). The service life of unimproved hot-poured and cold poured seals is considered to be about two years, while preformed materials can be expected to last from 5 to 10 years (8). Utah's experience indicates that improved hot-poured materials should have a

Table 6-2. Material Cost Data for a 24 ft (7 m) Long Joint Sealant, in 1974 Dollars.

Material	Joint Size, in. 3/8 x 1/2	1/2 x 3/4	3/4 x 3/4	1 x 1
Hot-Poured Rubberized Asphalt	$ 2.27	$ 0.90	$ 1.32	$ 2.38
Improved Hot-Poured Rubberized Asphalt	3.15	1.24	1.83	3.29
Polysulfide, Cold Poured	12.07	4.72	7.12	12.63
Urethane, Cold Poured	5.47	2.14	3.24	5.74
Neoprene, Preformed				24.19

1 in. = 2.54 cm

From Reference 13.

service life approaching the preformed sealants, provided proper joint design and construction are employed (14). However, according to Brown (15), the preformed joint seal made with neoprene is performing much better than the liquid sealed joints, primarily because of foreign matter being compressed into the liquid sealed joints, creating distress at the joints.

6.4.2 Horizontally Bored Sleeper Slab and Joint Restoration. On joints over granular bases where the pavement is overloaded causing spalling or faulting (often accompanied with sealant failures allowing water intrusion and thus pumping), an innovative technique is to strengthen the pavement structure in the vicinity of the joint by a horizontally-bored sleeper slab, followed by restoration of the joint sealing system (Figure 6-3). A horizontal hole is drilled from the side of the pavement directly under the joint (see Figure 3-2 in chapter 3). The hole is then filled with a strong, yet permeable, material (such as an open-graded HMAC) which will perform two functions. One, it will provide an escape path for any water leaking through the joint. And two, it will provide load transfer across the joint. After placement of the sleeper slab the joint should be cleaned out and resealed. The concept of providing a drainage layer under the pavement is not new. Georgia reports the use of a 3 in. (7.6 cm) open graded HMAC directly under the pavement (16) to provide an escape route for surface water entering the joints.

The cost or permanence of this technique is not known, as it has not been tried (to our knowledge).

6.4.3 Underseal by Pressure Injection. Pressure injection techniques are not new. As discussed in Chapter 3, the major problem with such techniques is to avoid uplifting the slab beyond what is required to level it. To prevent such an outcome the pressures must be kept low. However, slab jacking and slab undersealing have been successfully employed in a number of instances (17, 18, 19, 20, 21). Both asphalt and portland cement slurries have been used. Where leveling is desired portland cement grouts seem to perform better (20, 21), but when undersealing is the prime objective asphalt emulsions work quite well (17).

The cost of subsealing a slab is quite variable and little published information is available. In a New York report (17) the 1962 costs per mile (0.6 km) averaged $100,000 which is assumed to be for at least

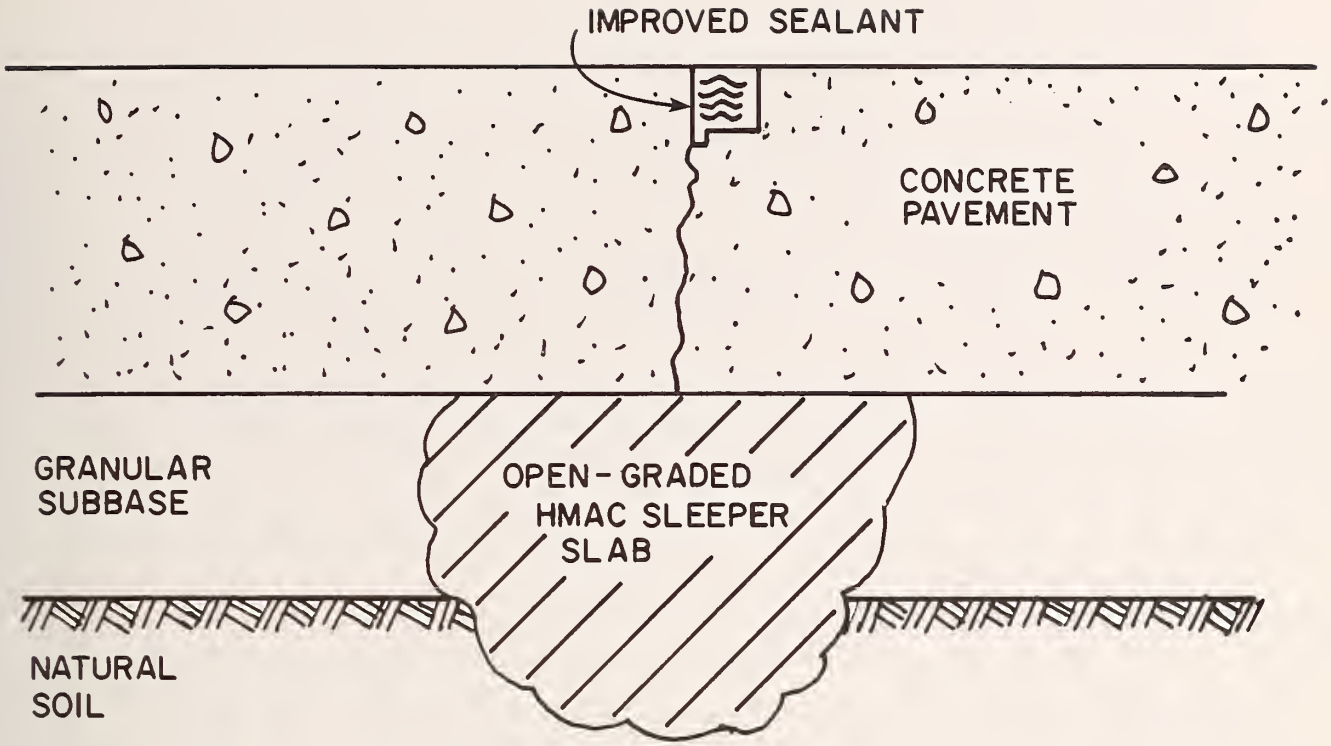


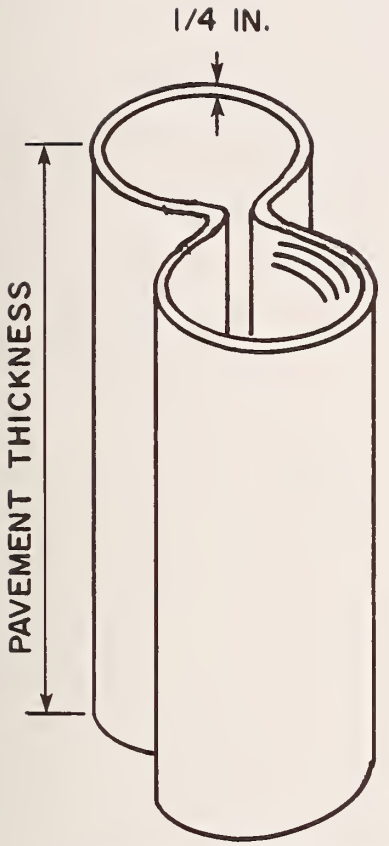
Figure 6-3. Horizontally-Bored Sleeper Slab.

two lanes. Equipment to inject the sealant is readily available and fully described in the literature (19, 20, 21).

6.4.4 Improved Load Transfer Devices. To date, the most effective load transfer devices seem to be aggregate interlock for short slabs and round dowels for longer slabs (5, 8, 12). The major problems with dowels are misalignment during construction and corrosion with time (5, 12). The result is often a joint failure. The design of dowels has been well developed (10), and thus, if the corrosion and construction problems can be overcome, joint performance should improve. Plastic coatings are being investigated for corrosion protection, and thus far are performing well (5). Misalignment can be reduced by careful inspection, strengthened tie downs, and tighter specifications (22). This will increase the cost of the joint, but the results may be worth the increased expense.

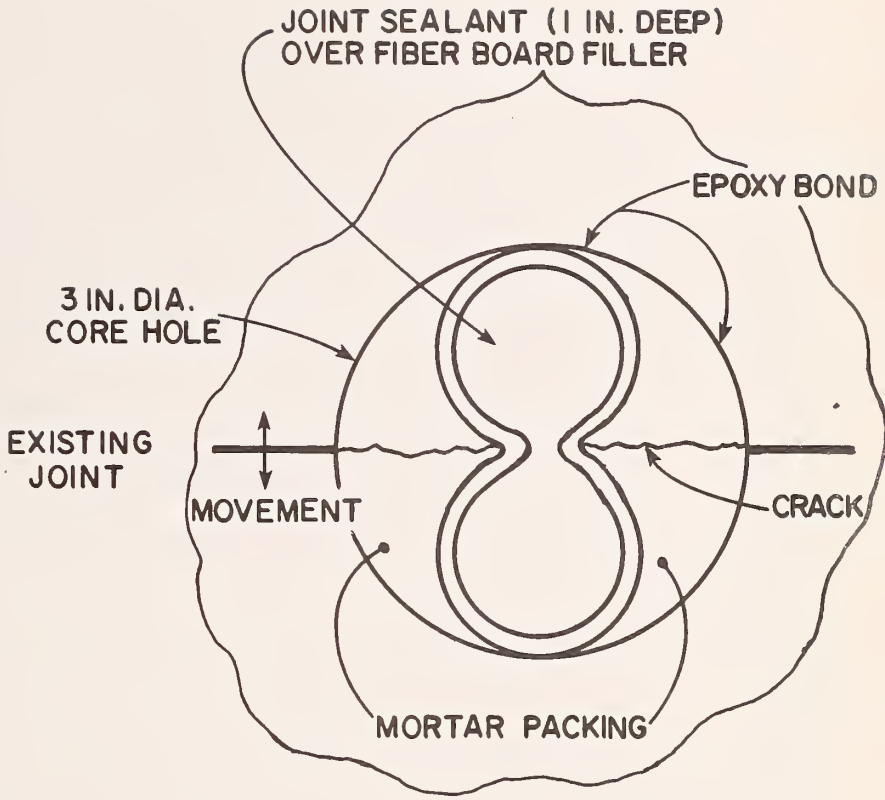
The major problems with aggregate interlock are keeping the joint sealed and keeping the crack width narrow (8, 12). This means keeping the transverse joint spacings at 15 ft (4.6 m), or less, in order to keep the cracks tight.

Two new load transfer devices are proposed. They are the "Figure Eight" device (Figure 6-4), and the "Vee" device (Figure 6-5). These devices are designed to be placed in an existing pavement in which additional load transfer is needed. For maximum effectiveness these devices should be installed when the pavement is as cool as practical (hopefully near to its coldest temperature during the year). At this temperature the pavement will be in its most contracted position and thus during all other times the devices should be in compression, which will increase load transfer. The "Figure Eight" device would come as a complete assumbly including the fiberboard filler and 1 in. (2.5 cm) deep sealant on the inside and the mortar packing on the outside of the device. First, 3 in. (7.6 cm) diameter cores would be cut at the joint for each lane. The inside edge of each cylindrical hole in the pavement would then be coated with epoxy cement and the device inserted. As soon as the epoxy hardened (around 30 min.) the pavement could be opened to traffic. Stainless steel should be used so as to minimize corrosion effects.

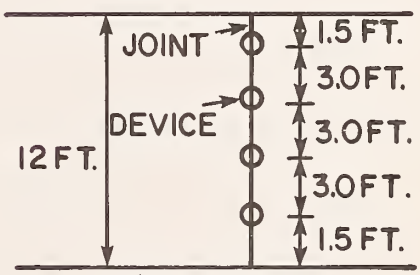


ISOMETRIC VIEW OF DEVICE

1 FT. = 0.3m
1 IN. = 2.54 cm

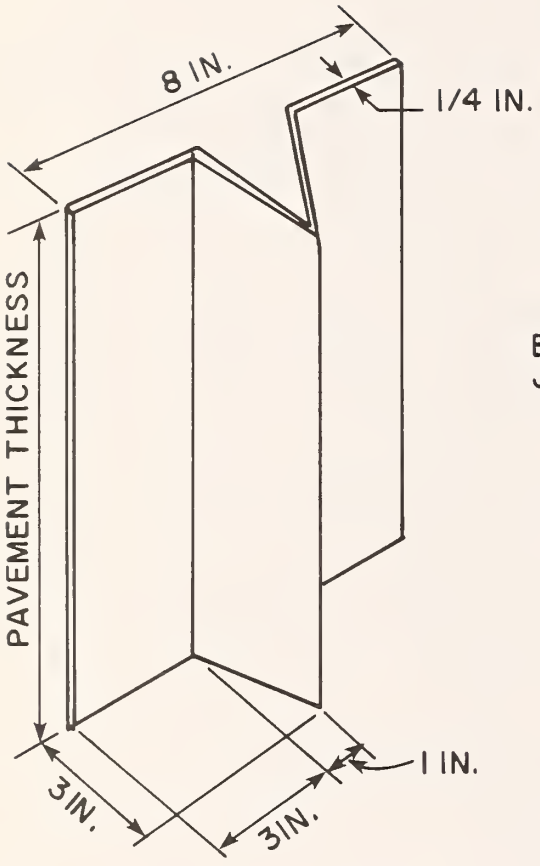


PLAN VIEW



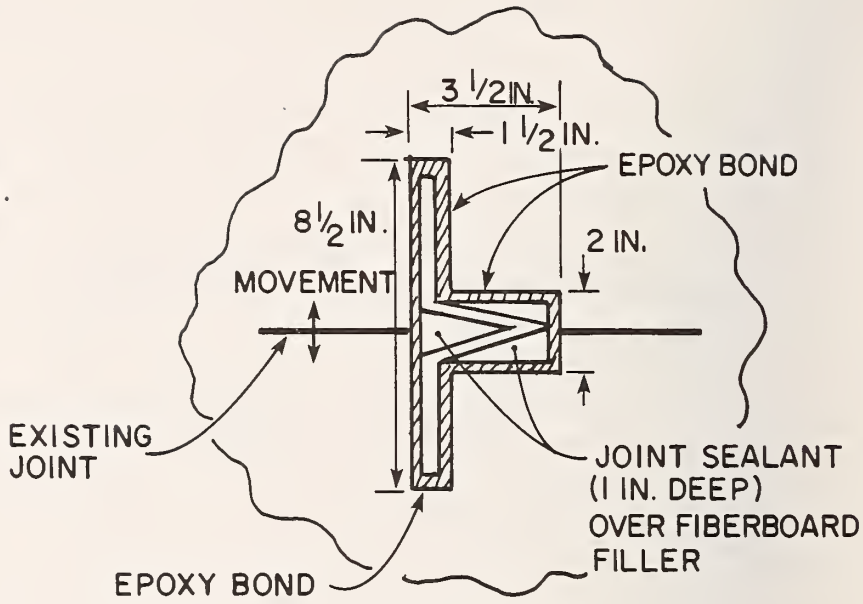
PLAN VIEW

Figure 6-4. Figure Eight Steel Load Transfer Device (Schematic).

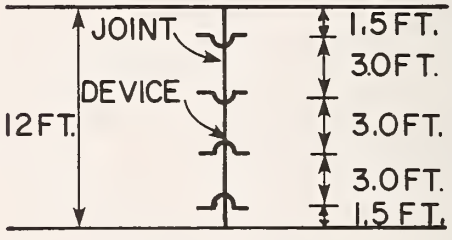


ISOMETRIC VIEW OF DEVICE

1 FT. = 0.3 m
1 IN. = 2.54 cm



PLAN VIEW



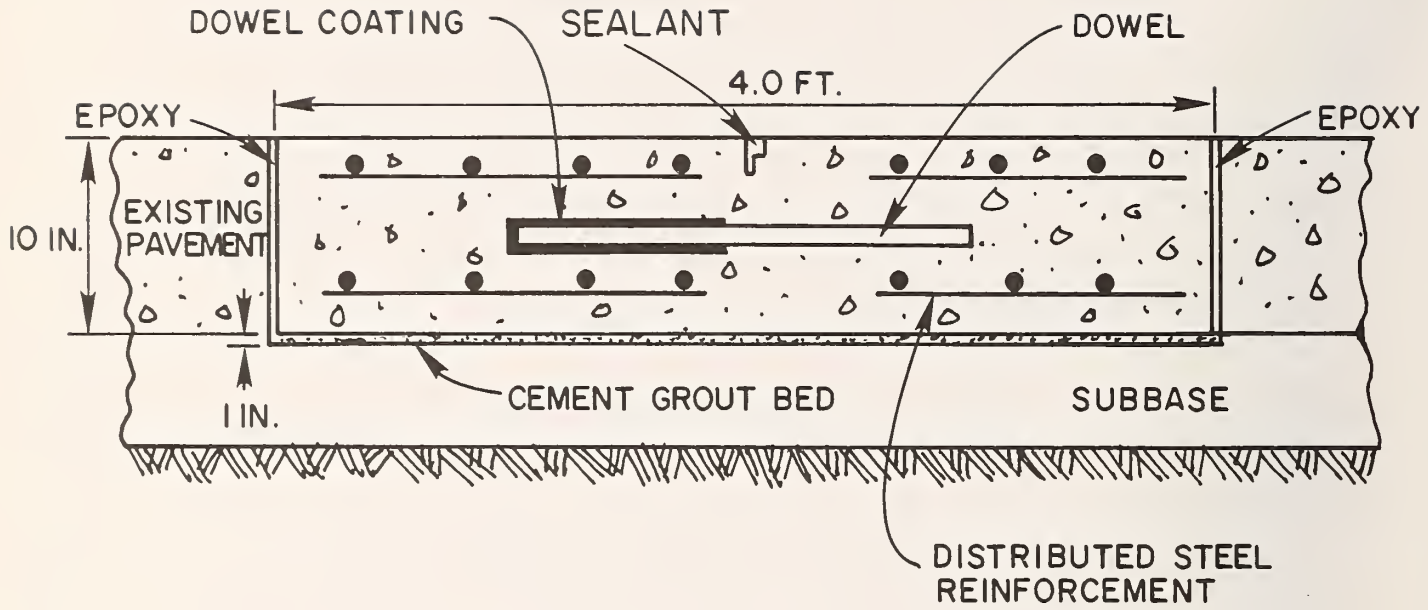
PLAN VIEW

Figure 6-5. Vee Load Transfer Device

The "Vee" device would also come as a complete assembly including the 1 in. (2.5 cm) deep sealant over the fiberboard filler. A ½ in. (1.3 cm) wide by 8½ in. (21.6 cm) long slot would be cut across the joint and a 2 in. (5 cm) wide by 3½ in. (8.9 cm) long slot would be cut along the joint (see Figure 6-4). The cuts could be made with high pressure water jets, such as the Flow Research, Inc. water jet cutting system using their hydraulic intensifier (23). Such a cutting system should be able to cut the desired dimensions with little or no difficulty (24). Following the cutting and cleaning, the vertical exposed faces of the cut would be coated with epoxy cement and the device installed. Once again the pavement could be reopened to traffic within 30 min. after installing the devices.

6.4.5 Precast Joint Assembly. This technique involves the use of precast joint assemblies to repair badly failed, spalled, and disintegrated joints from "blow up". There are two reasons to consider precast assemblies. One is that factory assembling of precast units permits very close construction tolerances. As discussed previously, one serious problem has been the misalignment of load transfer devices in the field. Dowel bars, for example, can be perfectly aligned and tied down before the slip form paver comes through, only to be shifted and misaligned by the fresh concrete as the paver passes over the joint. This happens altogether too often with the result that joints lock up and disintegrate, thus requiring replacement. A precast unit could be inspected for joint movement before installation. The second reason is that precast units permit reopening the roadway to traffic very quickly (30 min. after installation if epoxy is used). A complete discussion on precast elements is given in Chapter 8.

Two types of precast joint assemblies appear promising. One is the doweled joint assembly (Figure 6-6). Here the assembly is constructed, in a casting yard under close supervision, to preselected sizes. For example, the size shown is 4 ft (1.2 m) by 12 ft (3.7 m) by 10 in. (0.25 m) thick. The existing pavement would be cut, full depth, and removed to expose a 4.1 ft (1.25 m) long section of the subbase, 12 ft (3.7 m) wide. The subbase would then be cut down approximately 1 in. (2.5 cm). This could be followed by placing a 1 in. (2.5 cm) thick layer of portland



1 FT = 0.3 m
1 IN. = 2.54 cm

Figure 6-6. Precast Dowel Joint.

cement grout as a bedding layer (on top of a polyethylene sheet bond breaker if necessary). Next the precast joint assembly would be placed in the void and seated to proper elevation. The final step would be to bond the assembly to the existing pavement with a high strength epoxy cement.

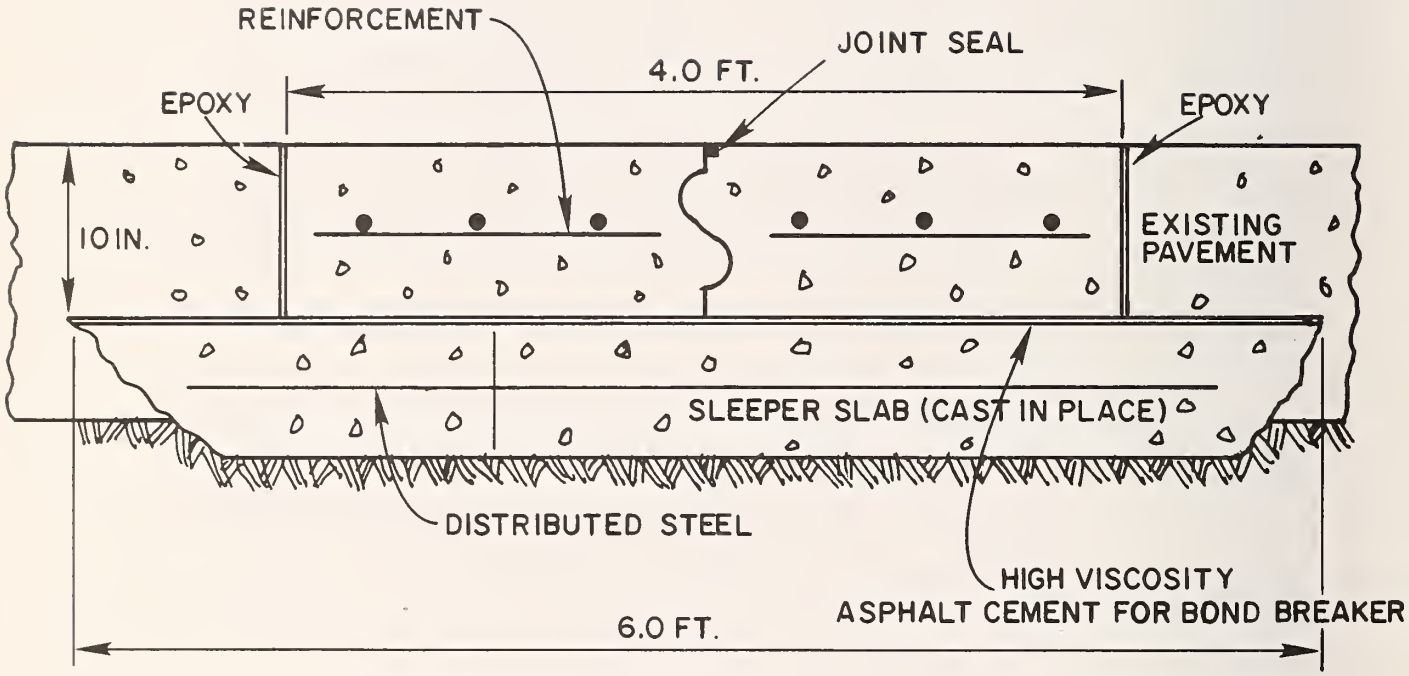
The second type is the precast supported-joint assembly (Figure 6-7). In this assembly, load transfer is provided by a cast-in-place sleeper slab and a contoured joint face (25). The precast assembly is cast in two peices with the contoured joint-faces cast against steel forms held to very close tolerances. The existing pavement is sawed, full depth, and removed to expose the subbase. The subbase, and subgrade if necessary, is removed to a depth of 6 in. (15 cm) as shown on the figure. Following removal the underside of the existing pavement is coated with a thick layer of high viscosity asphalt cement. Then preplaced aggregate concrete is utilized for the cast-in-place sleeper slab (25). By using preplaced aggregate and a cement grout, the entire void under the slab can be completely filled with concrete and there is no need to wait before placing the precast assembly. The slab surface is then finished and coated with a thick layer of high viscosity asphalt cement. This is immediately followed by placing the precast joint assembly and epoxy cement. Once the epoxy cement has hardened, the pavement may be opened to traffic.

As with the improved load transfer devices, these precast assemblies should be installed when the pavement is near its coldest temperature in the year. By so doing, the assembly should be in compression most of the time, which will aid in load transfer.

6.5 Utility of Potential Rehabilitation Techniques

In order to compare the utilities of the various techniques, the following general assumptions were made:

1. The pavements were all JRCP with 60 ft (18 m) joint spacings.
2. The pavements were all located in a suburban area, with about 5000 veh/lane day.
3. Segmental rehabilitation required all joints to be repaired while localized rehabilitation required 10% of the joints to be repaired.
4. For conditions other than blow up, the Serviceability Index (SI) of the pavement, at the time of rehabilitation, was 3.5 (thus making the



1 FT = 0.3m.
1 IN. = 2.54 cm.

Figure 6-7. Precast Support-Joint Assembly.

techniques preemptive rather than restorative), but without rehabilitation the SI would drop to 2.5 in from 3 to 8 years.

In addition to these general assumptions, each technique required some special assumptions. For example, faulting (where applicable) was assumed to be minor (around $^1/_4$ in. or 0.6 cm), and blowups (where applicable) were considered to affect a pavement length of 4 ft (1.2 m).

With these assumptions, the utility value for each technique was determined for certain representative conditions (Table 6-3). From these data, it appears that under certain conditions improved joint sealants, horizontally bored sleeper slabs, and the precast joint assembly may be significantly better than currently used techniques. Thus, their use should be investigated further, as indicated in Table 6-4. The improved load transfer devices (Figure Eight and Vee) had a lower utility than the horizontally-bored sleeper slab, but the difference was minor. Since none of these techniques have been tried and thus their utilities are based on uncertain input data, they are all worthy of continued consideration at this time. As discussed earlier, the joint problem is one that has plagued highway engineers from the beginning. Any potential technique that exhibits a utility even close to existing should be more fully evaluated.

Table 6-3. Joint Treatment Techniques to Solve Specific Problems

Technique \ Distress Manifestation	Raveling, Spalling, Pumping (Segmental)	Faulting, Keyway Failures (Segmental)	Compression Cracks (Localized)	Blowups (Localized)	Spalling, Pumping (Localized)
A. Potential Techniques					
1. Joint restoration with improved joint sealants	0.741				0.760
2. Horizontally-bored sleeper slab and joint restoration	0.757	0.757			
3. Underseal by pressure injection (see Chapter 3)	0.740	0.740			0.651
4. Improved load transfer devices (Figure Eight device or Vee device)	0.726	0.726			
5. Precast joint assembly					
a. Doweled joint assembly	0.738	0.738	0.729	0.729	0.729
b. Supported joint assembly	0.733	0.733	0.729	0.738	0.738
B. Current Techniques (Partial List)					
6. Clean and Reseal with cold poured polysulfide	0.676				0.676
7. Overlay with 2 in. (5 cm) HMAC	0.737	0.737			0.737
8. Digout and Repair (Localized)			0.655	0.655	0.650

Table 6-4. Summary of Promising Joint Treatment Techniques and Applicable Conditions

Potential Techniques	Applicable Conditions (Rigid Pvt.)
A. High Priority	
1. Joint restoration with improved joint sealants	Segmental and localized raveling, spalling and pumping
2. Horizontally-bored sleeper slab and joint restoration	Segmental raveling, spalling, pumping, and minor faulting
3. Precast joint assembly	Localized blowups, compression cracks, spalling, and pumping
B. Special Consideration	
4. Improved load transfer devices	Segmental and localized raveling, spalling, pumping, and minor faulting

7. SURFACE RESTORATION OF RIGID PAVEMENTS

7.1 Current Techniques

As PCC pavements age and are subjected to the abrasive, wearing action of hordes of vehicles, their surfaces are worn down. And in the northern climates, the addition of deicing salts during the winter accelerates surface deterioration. Even the best PCC pavements eventually wear down and the surface skid resistance is reduced to the point where some action must be taken. In addition, minor faulting and consolidation produce a rough ride with SI values dropping to unacceptable levels. Quite often the pavement is structurally sound with many, many years of useful life left. The question arises as to how to restore the surface to a higher SI value with a high skid value and maintain these values over a prolonged period. Currently the highway engineer has the following choices (other than complete replacement):

1. He may overlay the pavement with a relatively thick layer of asphaltic concrete or PC concrete. This is expensive and presumes vertical clearance and drainage will permit such thick overlays.

2. He may cut grooves in the pavement, using a series of diamond blades. This technique is relatively inexpensive and -- as long as the structure is sound -- affords an economical, long lasting solution; but it does not improve rideability. The technique is well developed, with a number of machines available on the market and an association actively promoting grooving (1). A number of states have grooved their PCC pavements with significant reductions in accidents (2, 3, 4). The structural capability of the pavements is not impaired so the life is not reduced. Currently, the 1977 cost of such grooving, in Texas, costs around \$0.80 to 0.90/yd^2 (\$1.00 to 1.10/m^2), assuming normal job size and traffic control measures (5).

3. He may level the ride and roughen up the pavement surface by removing approximately ¼ in. (0.6 cm) of the surface. Currently there are at least three machines which will remove a thin layer of PC concrete, leaving a leveled surface with a very high skid resistance. They are the Klarcrete machines manufactured by Klarcrete Limited (6), the Pavement Profiler, manufactured by CMI Corporation (7), and the power

planer, manufactured by the Barco Manufacturing Co. (8). The Klarcrete machine has been used to remove up to ¼ in. (0.6 cm) of the old surface and also to clean old surface for overlays (9, 10). The CMI profiler, a new machine, is reported to be able to remove up to ½ in. (1.3 cm) of PCC pavement, 9 ft. (2.7 in.) wide at a rate of about 20 fpm (0.1 m/s) for a 1976 cost of around $3.00/yd^2 ($3.60/m^2) (11). Currently in Texas, shallow surface roughening of PCC pavement costs around $0.50 to $0.75/yd^2 ($0.55 to 0.80/m^2), assuming normal conditions (5).

Each of these solutions are well developed and successful under certain conditions. Techniques two and three (grooving and leveling/roughening) are examples of surface restoration without the use of overlays and thus are applicable for investigation on this project. The main innovative features to them would be the introduction of more efficient machines to groove or roughen the pavement surface.

7.2 Proposed Techniques

As the current methods of surface rehabilitation are, in general, quite good, the only innovative technique uncovered on this study was the combining of a pavement leveling/roughening technique with either a thin overlay or surface strengthening to provide longer lasting, high skid resistance. Following a cleaning and leveling action with one of the existing machines, a high strength, thin surface could possibly be obtained by:

1. Addition of a super water reduced concrete pavement surface.
2. Addition of a polymer-modified concrete pavement surface.
3. Addition of a fiber reinforced concrete pavement surface.
4. Polymer impregnation of the pavement surface.

Each of these techniques will be discussed in the following paragraphs.

7.2.1 Super Water Reduced Concrete. Certain admixtures, when added in relatively large quantities to PC concrete, permit reduction of 15 percent or more water without the loss of workability. This reduced water content results in significantly higher strengths (both flexural and compressive) which, in turn, will permit thinner overlays to be used (12). However, one problem has developed. The use of certain admixtures results in reduced freeze-thaw resistance, so the long term durability of these mixtures is in doubt (13). The Federal Highway Administration

is currently sponsoring research on these super water reducers, so no judgement can be made at this time as to their overall potential. If they can be shown to be durable, then the increased strength may offset the costs involved for the admixture. As the question of durability remains to be answered, this technique is not considered further in this report.

7.2.2 Polymer Modified Concrete. Polymer modified concrete is a concrete where a liquid polymeric material is introduced into the mixer along with aggregates, portland cement, and water; the polymeric material is subsequently hardened. The polymer modified concrete, with its excellent strength and durability properties have potential application for thin overlays. De Puy (14) reports that epoxy, furan resins, and methyl methacrylate and styrine monomer have been successfully utilized. Compressive strengths range from 12000 to 24000 psi (83,000 to 165,000 kPa). He notes that polymer modified concrete is versatile and may be conveniently adapted to many uses (14). The Naval Civil Engineering Laboratory (15) has investigated polymer modified concretes using one epoxy (Celanese Epi-res 5077) with one curing agent and a saran latex (DOW 464), which is a milk-like emulsion consisting of small globules of the saran plastic dispersed in water. They reported:

"1. For situations demanding high strengths at an early age (1 to 7 days), polymer-modified concretes made with epoxy resin should be used.

2. For designs requiring relatively high strengths at 28 days and beyond and in which early high strengths are not so important, latex-modified concretes should be chosen because they are cheaper and easier to use.

3. Generally speaking, compared to similar conventional concrete, properly prepared and cured polymer-modified concretes in which the polymeric materials are added at the mixer can be relied upon to provide increased compressive, splitting tensile, and flexural strengths, adequate bond strength and Young's modulus, and reduced water absorption."

They further report:

"12. Consistency of epoxy-modified concretes is extremely thick and sticky and the material is difficult to place (compact). Consistency of latex-modified concrete is good and it is easily compacted.

13. When making epoxy-modified concrete with portland cement (PPCC), the basic portland cement concrete mixture must be allowed to stand for 1 hour prior to adding the epoxy; with an accelerator, this time can be reduced to 15 minutes. When epoxy-modified concretes are made with Regulated Set cement (PRCC), no waiting period is necessary.

15. Generally speaking, consistently highest strengths were obtained over all test ages (1) with 75% epoxy loading in PPCC or PRCC and (2) with 40% latex loading in PPCC. (Loading is based on a percent of cement weight.)

17. PPCC with 75% epoxy loading of Epi-res 5077 costs $\$465.37/yd^3$ more than conventional concrete; the same loading and mix with Epon cost $433.20 yd more than conventional concrete.

19. Highest compressive strengths and highest flexural strengths were obtained with epoxy-modified concrete made with PRCC; highest splitting tensile strengths were obtained with epoxy-modified concrete made with portland cement."

Note that epoxy modified concrete must be allowed to stand for one hour before adding the epoxy, to provide for some hydration. Polymer addition too early impedes hydration and results in insufficient strength. Twenty-eight-day compressive strengths of 8390 psi (57800 kPa) and splitting tensile strengths of 1190 psi (8200 kPa) where achieved with a 5.9 sk/yd^3 (330 kg/m^3) concrete (15). Thus thin overlays, although expensive, may be feasible using this technique.

7.2.3 Fiber Reinforced Concrete. Fiber reinforced concrete, defined as "concrete made of hydraulic cements containing fine or fine and coarse aggregate and discontinuous discrete fibers" (16), have been the subject of extensive research in recent years. The state-of-the-art has been clearly and concisely summarized by ACI Committee 544, with 101 references (16)

and by the Concrete Society in England with 80 references (17). A summary of the current state-of-the-art is given in an NCHRP report (18), and -- for completeness -- is reproduced here.

Portland cement concrete, when used as a pavement material, is generally regarded as having one significant disadvantage -- a low tensile strength in comparison to its compressive strength. This low tensile capacity can be significantly increased by the addition of discrete, randomly-oriented fibers which act as crack arrestors and significantly increase the ultimate tensile load-carrying capacity of the concrete. When the fiber's volumes are kept reasonably low -- say less than around 4 percent by volume -- the concrete's proportional limit does not seem to be significantly increased, but its ultimate flexural strength can be more than doubled (19). There seems to be no quantitative expression yet for the prediction of the strength of the fiber-reinforced concrete as a function of the strength of the plain concrete, strength of the fibers, fiber volume, fiber geometry, etc. However, experimental data do indicate the significant improvement of ultimate strength in either flexure or indirect tension, which is very important for a pavement material.

Fibers have been produced from steel, plastic, glass, and natural materials in various shapes and sizes (17). Of these, steel and glass have been found to be the most useful and cost effective. In the following paragraphs each of these fibers will be discussed in terms of the current state-of-the-art.

Steel Fibers. Typical aspect ratios (fiber length divided by equivalent fiber diameter) range from 30 to 150 for lengths of 0.25 to 3 in. (6.35 mm to 76.2 mm). Both round and flat steel fibers have been produced and used, as well as crimped and deformed steel fibers (17). Various types of steels have been used with tensile strengths from 40 to 600 ksi (275 MPa to 4,100 MPa).

Steel fibers have a tendency to ball or clump together in the mixer, thus mixing has been a problem. Due to these difficulties the aspect ratio should be limited to less than about 100 and fiber contents should be limited to less than around 2 percent (17). Furthermore, the maximum recommended coarse aggregate size is about 3/8 in. (10 mm). ACI Committee 544 has made specific recommendations about the mixing and placing procedures to be followed to reduce segregation and promote proper coating of the fibers with cement paste (17).

Several different applications of steel fiber concrete have been utilized, including a number of pavement applications. Excellent summaries of pavement applications have been reported by Lankard and Walker (20), Gray, et. al. (21), and Pasko (22). Problems do exist in the use of steel fiber concrete. According to Pasko (22):

> "These problems can be crudely grouped into three major areas; economics, materials and structural.
>
> Economically, the cost per cubic yard of fiber reinforced concrete is high primarily because of the steel cost and secondly because of the high cement content, smaller aggregates, and the extra materials

to be handled such as fly ash, water reducers, and steel. The steel itself presently costs about 18 cents per pound (40 cents per kg) which, with greater demand and the use of scrap, may possibly decrease to 10 cents per pound (22 cents per kg) in the future. Tremendous gains have already been made when one remembers that several years ago the cost was 65 cents per pound (143 cents per kg) for this specialty product. Based on laboratory tests, the amount of steel necessary in the concrete appears to be between 1 and 2 percent by volume which is 133 to 265 lbs per cu yd (78 to 157 kg per m^3). A lesser amount does not provide much advantage over plain concrete and greater amounts are less effective and may cause mixing difficulties.

By comparison, a continuously reinforced concrete pavement 8 in. (200 mm) thick with a cross sectional percentage of steel of 0.64 percent (No. 5 bars on 6 in. (150 mm) centers) contains only 85 lbs per cu yd (50 kg per m^3) or about 1/3 of that in fiber reinforced concrete containing 2 percent steel by volume.

Hopefully, through the use of different sizes and shapes of fibers and new cements it may be possible to reduce the amount of steel, but this is not imminent at this time.

In regards to materials, some difficulties still exist, but great progress is being made in resolving them . . . Handling and dispersing the steel is still difficult but this could be easily overcome by developing special purpose equipment. Similarly, some changes may possibly be needed in the paving train, particularly in vibration, to improve the placement of stiffer mixtures. Some development work has already been started on building a device to automatically convey the fibers into the mixer.

While the difficulties with materials are slowly being ironed out, the structural design of fiber reinforced concrete pavements has largely been neglected to date. Much information is presently available on regular concrete pavements which can be judiciously applied to evolve designs with which the slabs could be built to resist the ravages of environment and loads.

The limited work done at the Waterway Experiment Station both in load testing and theoretical analysis indicate that the thickness of fiber reinforced concrete slabs-on-grade and overlay should be roughly one-half of the thickness required for plain concrete slabs. Although this is a good starting point it will have to be further substantiated and defined for other conditions.

> The design of jointing has yet to be resolved. . . . If the cracking is allowed to randomly occur and form irregular, erratic patterns, traffic will soon cause spalling and raveling of the concrete faces which will create rough riding, unsealable cracks. Water and debris can then infiltrate the cracks and lead to pumping, faulting and blowups.
>
> It is therefore necessary that a joint pattern be superimposed on the fiber reinforced concrete ribbon to control the cracking. Also, since the slab ends are the weakest point of a slab it is necessary to provide some type of continuity at the cracks or joints such as dowel bars or other devices."

Glass Fibers. Considerable research has been conducted on the use of glass fibers in concrete (17, 21, 23, 24). Typical glass fibers have diameters of 0.2 to 0.6 in. (0.5 to 15 cm), but these fibers may be bonded together to produce glass fiber bundles with diameters of 0.05 to 0.50 in. (0.013 to 1.3 cm) (17). Lengths of the glass fibers are similar to steel fibers (0.25 to 3 in. (6 to 76 mm), so their aspect ratios range from about 30 to 150. The glasses used for these fibers have tensile strengths from 150 to 550 ksi (1,000 MPa to 3,800 MPa). Several glass fiber compositions are in use today, two of which are in general production (17). They are the soda-lime-silica glass (A glass) and the borosilicate glass (E glass). Proprietary treatments have been used to prevent detrimental chemical reactions between the glass and the alkalies in the portland cement (similar to alkali-silica aggregate reactions) (16).

Applications of glass fiber concrete in pavements have been limited. One field test section has been reported (25, 26), with results encouraging thus far. The glass fibers were added at the job site, in a ready-mix truck, and thoroughly distributed throughout the mix in volume percentages of 1 and 1.5. No segregation or balling was observed. The only problem reported was that initially the concrete was very stiff after the addition of the fibers, and a higher water/cement ratio had to be used to provide workability. Another field application was conducted in Minnesota and some construction problems were reported.

Analysis. From the experience to date, it appears that fiber-reinforced concrete offers considerable potential for producing pavement sections much thinner than regular portland cement concrete. Practically speaking, there are problems with the construction and cost of fiber-reinforced concrete. These need to be overcome before the concept will be utilized to any appreciable extent.

If a thin overlay is deemed necessary, then fiber reinforced concrete might well be used.

7.2.4 Polymer Impregnation of the Pavement Surface. This technique, termed polymer impregnated concrete (PIC), involves the drying of the

pavement surface layer, then impregnating the dried volume with a monomer -- which is then polymerized -- to effect a stronger and more durable concrete. Manson, et al., have prepared an extensive report on the use of polymer impregnation in highway concrete as part of NCHRP project 18-2 (27). This report is currently pending publication by NCHRP. They report (27):

> Using gas-fired infra-red heaters for drying the concrete and moderate pressure impregnation followed by hot-water polymerization, it was demonstrated that deep (up to 4 in. or 10 cm) polymer impregnation could be achieved under field conditions. Demonstration projects were carried out on two bridge decks, both structurally sound and one severely salt contaminated. The equipment and techniques developed for and used in these demonstrations are suitable for scale-up for commercial applications.
>
> A field manual describing the techniques and including safety precautions and acceptance criteria was developed and is included in the final report on the project.

The cost of this technique, as reported in 1975 dollars by Manson, et al., (27) are summarized in Table 7-1. These are direct costs and do not reflect capital equipment expenditures. One lane mile (1.6 km) of pavement impregnated 2 in. deep (5 cm) would cost around $160,000 (in 1975 dollars) if the unit cost per ft^2 (.09 m^2) of $2.507 is assumed valid. This unit cost is based on a 12 x 40 ft (3.7 x 12.2 m) slab size, and certainly the economics of scale would effect some cost savings. However, the cost would still be high.

Cost aside, there is another factor which will significantly influence the utility of polymer impregnation; the time required. To dry, impregnate, and polymerize a 12 x 40 ft (3.7 x 12.2 m) slab, Manson, et al., (27) estimate 5 to 6 weeks of 24 hr a day operation. This time is for a 4 in. (10 cm) impregnation. If the time could be reduced to 2 weeks for a 2 in. (5 cm) impregnation, one lane mile (1.6 km) would still take 264 weeks (or 5 years)! Thus a number of heaters, impregnation units, and personnel would be required to reduce the construction time to anywhere near acceptable values.

A third problem with PIC is the large quantities of energy required to dry the pavement. Approximately 16,000 Btu/ft^2 (1.82 x 1.0^8 J/m^2) are

Table 7-1. Total Costs for Polymer Impregnated Bridge Decks

Specified Impregnation Depth, in.	Drying Cost $/ft^2	Impregnation Cost $/ft^2 @0 psig	Impregnation Cost $/ft^2 @15 psig	Polymerization Cost $/ft^2	Total Cost $/ft^2 @0 psig	Total Cost $/ft^2 @15 psig
1	0.216	1.000	0.875	0.176	1.302	1.177
2	0.258	2.208	1.792	0.457	2.923	2.507
3	0.416	3.583	2.758	0.923	4.922	4.089
4	0.614	5.125	3.792	1.577	7.316	5.983

Unit Conversions: 1 ft^2 = .0929 m^2

1 psig = 6.895 kPa

1 in. = 2.54 cm.

From Reference 27.

required to dry the pavement to a depth of 2 in. (5 cm) (27).

Therefore, on the basis of its high cost, long construction time, and excessive use of energy, PIC was not considered to be a potential technique for pavement rehabilitation.

7.3 Utility of Potential Rehabilitation

To assess the utility of the proposed techniques the following conditions were assumed:

1. Suburban conditions, with a current ADT of from 3000 to 6500 vehicles/lane-day.

2. The pavement has a serviceability index of 3.0, which will drop to around 2.4 within 4 years (primarily due to loss of skid resistance).

3. The leveling/roughening can be accomplished by removing approximately ¼ in. (0.6 cm) of concrete surface.

4. The joints and pavement structure are in good condition, and will remain so for at least 15 more years.

The resulting overall utility values for each technique are given in Table 7-2. For the assumed conditions neither of the two potential techniques appear as promising as the current technique of just roughening and leveling the pavement surface. Both polymer modified concrete and fiber reinforced concrete are extremely expensive and energy extensive. And even though it is estimated that the rehabilitated surface may last as long as 18 more years without further treatment, this long life does not overcome the expense and energy consumption involved with these two techniques.

Therefore, at this time, these techniques for surface rehabilitation of PCC pavements are not recommended for further consideration.

Table 7-2. Utility Values of Potential Techniques

Technique	Utility Values
A. Potential Techniques: Level/Roughen followed by:	
Polymer Modified Concrete Surface Course	0.690
Fiber Reinforced Concrete Surface Course	0.685
B. Present Techniques Level/Roughen the Surface	0.761

8. PRECAST CONCRETE ELEMENTS IN PAVEMENT REHABILITATION

8.1 Introduction

The prefabrication of structural and architectural elements in the building construction industry is an accomplished fact. In many applications the technique has proven its worth by improving product quality, decreasing time and cost of on-site construction, substantially decreasing environmental problems and providing a means of design and construction flexibility. The technique utilizing application in the United States dates back to 1906 when a two story building was constructed in Des Moines, Iowa, by casting the walls on the ground and then raising them into their final position (1). Other projects soon followed and by 1912 the precast concrete technique of building construction was an accepted practice.

Precast portland cement concrete elements have been successfully used in construction and rehabilitation of rigid pavements for many years. An early application was in the construction of a 1380 ft (453m) extension of the main runway at Orly Airfield near Paris (2), in 1945. The construction technique employed combined precast concrete elements with prestressing tendons to produce a pavement that performed satisfactorily under heavy loading conditions (3). Extensive load/deflection tests conducted on the Orly Airfield pavement resulted in only minor residual deflection when subjected to loads in excess of 140 tons (1.2×10^5kg) (4).

By 1960, approximately 31 prestressed highway pavements and 26 prestressed airport pavements had been constructed using post tensioning techniques.

The use of precast elements in rigid pavement rehabilitation has been limited largely to experimental projects. These projects have been conducted by various highway maintenance agencies and have resulted in the development of several techniques using precast elements ranging in size from small partial depth patching slabs (5), to full depth prestressed slabs weighing up to 22 tons (2.0×10^4kg) (6). In all applications the performance of the precast elements has been satisfactory but in most cases the cost as traditionally reported has been excessive.

One such project conducted by the Michigan Department of State Highways and Transportation in 1969 (7, 8), precipitated further research conducted in cooperation with the Federal Highway Administration (9), which resulted in the

development of standard design and construction specifications (10). The objective of this research project was:

> To develop procedures for replacement of distressed joints with permanent type repairs that can be opened to traffic within a few hours after installation.

The project afforded an opportunity to compare precast and cast-in-place pavement repairs through the study of 22 precast and 38 cast-in-place repairs installed on IH 96 and IH 69 near Lansing. The project showed both types perform well and that the precast repairs could be completed in approximately 25% of the time required for the cast-in-place repairs, but resulted in a cost increase of about 25%. This research effort as well as other experimental projects show that precast pavement rehabilitation is potentially feasible in areas where the level of user demand dictates minimum periods of normal traffic disruption.

The applications of precast elements in rigid pavement rehabilitation are briefly discussed in the following paragraphs.

8.2 Partial Depth Patching Slabs

Often a distress in rigid pavement is manifested only near the pavement surface. This is the case with spalling as it occurs at joints. Spalling is the result of joints failing to perform the task of compensating for the volumetric changes of the concrete due to changes in temperature. Failure occurs for various reasons as discussed in Chapter 6 but most often due to poor joint maintenance. Spalling typically is the deterioration of the slab edges at joints usually resulting in the removal of normally sound concrete. In the rehabilitation of this or any type of distress it is necessary to remove only the deteriorated concrete to facilitate bonding of the repair to structurally sound material. A rehabilitation technique applicable to this type of distress is the installation of a partial depth precast concrete patch.

The feasibility of partial depth precast patching slabs was demonstrated in 1971 by the New York Transportation Administration (11). Two 2 in. (50 mm) thick patching slabs approximately 3 ft^2 (0.3 m^2) each were installed in the Queen's Midtown Tunnel in six hours. In 1974, the Virginia Department of Highways and Transportation developed and experimentally implemented a similar rehabilitation technique.

Partial depth patching slabs were installed in the westbound lane of Va-44, the Virginia Beach-Norfolk Expressway (5). The original pavement was constructed in 1968 as a 9 in. (23 cm) thick jointed reinforced pavement with a joint spacing of 61.5 ft (18.8 m). The pavement distress was evident as spalling at the joints due to failure of the joint sealant material and intrusion of non-compressible materials.

Patch sites were prepared by removal of the distressed concrete using a Klarcrete concrete cutting machine. The Klarcrete general purpose machine used on this project consisted of eleven cutting heads 2 1/4 in. (5.7 cm) in diameter located 2 1/4 in. (5.7 cm) apart. The cutting heads were mounted on a carriage permitting transverse movement across the width of the area to be removed. Longitudinal movement was provided by relocation of the machine. Table 8-1 lists the technical and physical details of the Klarcrete general purpose machine. The deteriorated concrete was removed as required to reach sound material and provide space for the precast patching slabs and bedding grout. Typical patch details are shown in Figure 3.

Two types of precast slabs were produced for installation in this project. Portland cement concrete slabs were cast using a hydraulic press technique. They ranged in size from 2 ft x 3 ft x 2 in. thick (0.6 m x 0.9 m x 5 cm) to 1 ft x 2 ft x 2 in. thick (0.3 m x 0.6 m x 5 cm). Three of these slabs were cast using polymer impregnated concrete to test the resistance of this material to salt action. A 9 to 1 mixture of methyl methocrylate and trimethylopropane trimethacrylate and 1 percent by weight of vazo 52 solution was used as the treatment. The other type slabs were cast in the conventional manner using metal fibers for additional strength. The size of these slabs ranged from 2 ft x 2 ft x 2 in. thick (0.6 m x 0.6 m x 5 cm) to 1 ft x 1 ft x 2 in. thick (0.3 m x 0.3 m x 5 cm). The quantities of materials specified for this project are given in Table 8-2. A total of sixty-eight precast patches with a total surface area of 292.5 ft^2 (27.5 m^2) were installed in an average time of 76.8 min per patch including the time required for moving and setting up the Klarcrete cutter and removal of the deteriorated concrete. Table 8-3 summarizes the time requirements for the precast patches and Table 8-4 shows the average patching time distribution.

The factor controlling the production rate of this technique is the

Table 8-1. Technical detials of Klarcrete general purpose concrete repair machine.

Item	Value
Overall length	14 ft (4.27 m)
Length without tiller	10 ft 4.5 in. (3.16 m)
Width	7 ft 6 in. (2.29 m)
Overall height	4 ft 6 in. (1.37 m)
Weight	4,000 lb (1800 kg)
Power required, compressed air[a]	600 cfm at 100 psi (0.28 m^3/s at 689 kPa)
Cutting heads[b]	
Diameter	2 1/4 in. (57.15 mm)
Distance between heads	2 1/4 in. (57.15 mm)
Air consumption per head	30-35 cfm at 100 psi (0.014-0.017 m^3/s at 689 kPa)
Life	100 hours ± 25
Strokes per minute	1,500
Maximum cutting depth	4 in. (102 mm)
Accuracy	± 1/8 in. (±3.18 mm)
Width of cut	4.5 to 49.5 in. (0.114 to 1.26 m)

[a]The Klarcrete machine is pneumatically and hydraulically operated and pneumatically controlled on a sequential system.

[b]There are 11 tungsten carbide cutting heads, composed of a motor and a head; each head (except the first one) can be controlled individually. The cutting face is made up of 6 tips intersecting in the center of the head at 60 degrees. Depth of the cut depends on forward speed of the machine.

From Reference (5).

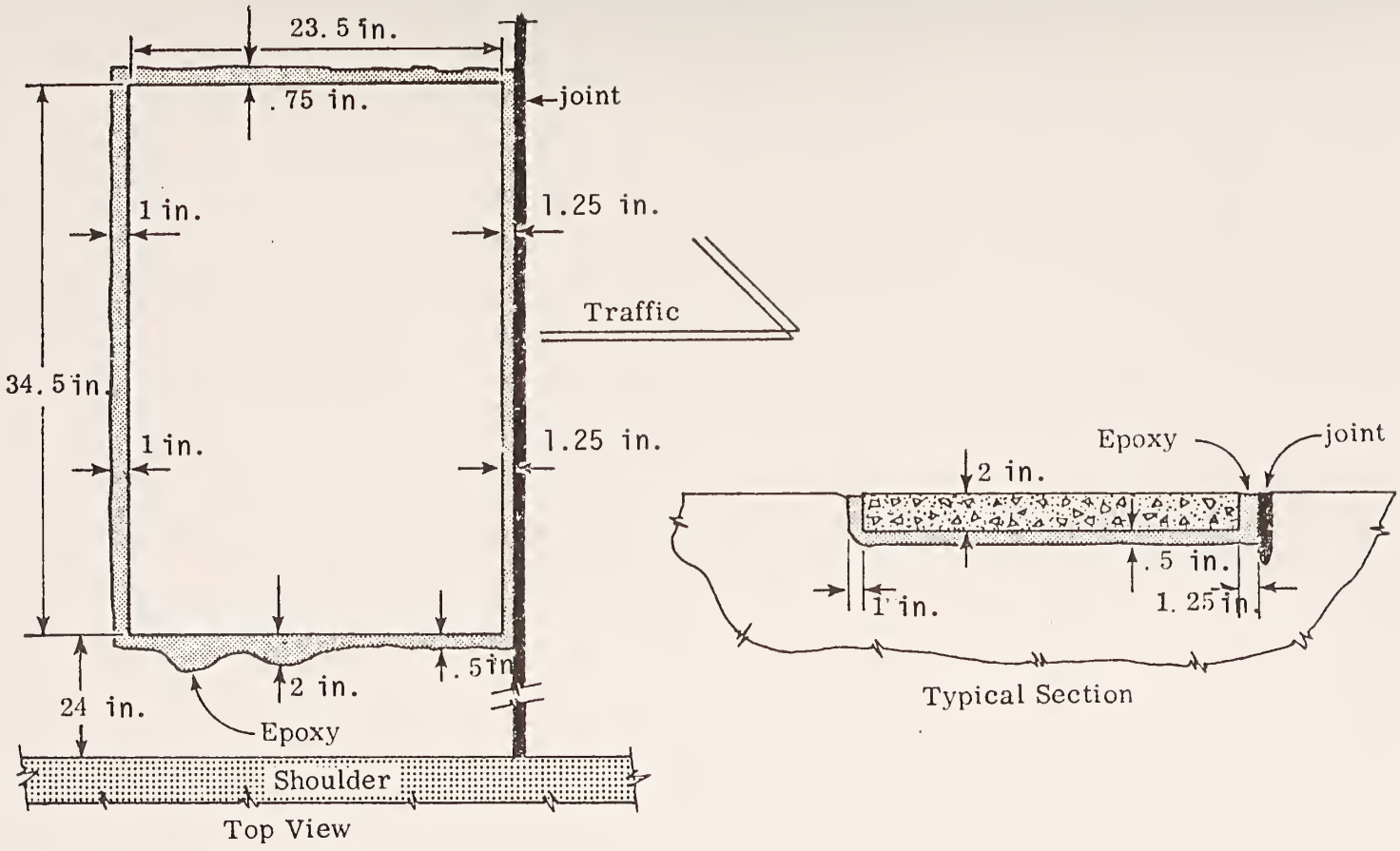


Figure 8-1. Sketch of patch showing dimensions of cut around patch.

From Reference (5).

1 in. = 2.54 cm.

Table 8-2. Quantities specified in contract for experiment.

Route 44 Pavement Repairs -- West Bound Lanes

From: Parks Avenue

To: 3.0 Miles West of Parks Avenue

ITEMS	QUANTITY
Removal Pavement 2 in. Depth	408 ft^2
Removal Pavement 4 in. Depth	20 ft^2
Removal Pavement 2 in. & 4 in. Depth	150 ft^2
Partial-depth Patch 4 ft. x 6 ft. x 2 in.	2
Partial-depth Patch 3 ft. x 4 ft. x 4 in.	1
Partial-depth Patch 2 ft. x 2 ft. x 4 in.	1
Partial-depth Patch 1 ft. x 4 ft. x 2 in.	1
Partial-depth Patch 4 in. x 3 in. x 2 in.	10
Partial-depth Patch 2 ft. x 2 ft. x 2 in.	25
Partial-depth Patch 1 ft. x 2 ft. x 2 in.	10
Wirand Patch — Installation (2 ft. x 2 ft. x 2 in.)	10
Wirand Patch — Installation (1 ft. x 2 ft. x 2 in.)	10
Cast-In-Place Joint Spall and Pavement Spall Repair	150 ft^2

1 in. = 2.54 mm; 1 ft^2 = 0.093 m^2; 1 mi = 1.6 km

From Reference (5).

Table 8-3. Summary of precast patches installed.

Number of Patches	Size*	Average Installation Time	Total Area	Total Time
1	1 ft x 1 ft	56 min.	1 ft^2	56 min.
15	1 ft x 2 ft	61 min.	30 ft^2	915 min.
1	1 ft x 7 ft	146 min.	7 ft^2	146 min.
1	1 ft x 10 ft	220 min.	10 ft^2	220 min.
14	1.5 ft x 2 ft	61 min.	42 ft^2	854 min.
1	1.5 ft x 3 ft	112 min.	4.5 ft^2	112 min.
1	1.5 ft x 4 ft	102 min.	6 ft^2	102 min.
14	2 ft x 2 ft	72 min.	56 ft^2	1,008 min.
14	2 ft x 3 ft	87 min.	84 ft^2	1,218 min.
5	2 ft x 4 ft	92 min.	40 ft^2	460 min.
1	3 ft x 4 ft	133 min.	12 ft^2	133 min.
68			292.5 ft^2	5,224 min. = 88 hours

*Each at 2 in. (5 cm) thickness

1 ft. - .3m
1 ft^2 - .093m^2

Table 8-4. Average times for patching operations

Operation	Minutes	Percentage
Machine set up time	17.4	23
Hole cutting time	41.6	54
Mixing epoxy and installing patch	17.8	23
Total	76.5	100

As shown in Table 8-3 a total of 292.5 ft^2 (27.5 m^2) of precast patches were installed in 88 hours.

1 ft = 0.3 m
1 ft^2 = 0.093 m^2

From Reference (5).

time required to prepare the site for installation of the precast slab. From Table 8-4, of the 76.5 min required to complete a repair, 41.6 min or 54 percent of the time is utilized by the hole cutting operation. Based on this average time, removal of the 292.5 ft^2 (27 m^2) of pavement required 36.6 hr for a production rate of 0.13 ft^2/min (0.01 m^2 / min). It is obvious that to achieve a higher rate of production more than one Klarcrete cutting machine or an alternative method of site preparation will be required. Figure 8-2 shows the relationship between the number of patches installed and the time of traffic disruption for 1, 3 or 10 Klarcrete cutting units.

Alternative methods include jack hammers, the CMI Rotomill and the Flow Industries high pressure water jet. The jackhammer was used on the Queen's Midtown Tunnel in which approximately 6 ft^2 (0.6 m^2) of precast patch was installed in 6 hrs. (11). This method is not as fast as the Klarcrete method and potential problems include accurate depth control over the area, control of the hole size and slope of the sides and the possibility of creating small fractures into sound concrete below and around the patch areas that could later cause further distress. The CMI Rotomill is designed for scarification of large areas of pavement (see ch. 7) and, thus, has limited potential as a feasible patching technique due to the generally small size of the area to be repaired. Also the CMI Rotomill does not produce square corners at the edges of the scarified area as required for this technique. The Flow Industries high pressure water jet system offers some potential even though the technique as required for partial depth patching has not been fully developed. Similar to the Klarcrete concrete cutter the water jet system consists of multiple rotating nozzles each removing approximately a 2.3 in. (60 mm) width of concrete (12). The depth of the cut is determined by the number of passes over the area. Figure 8-3 describes this relationship.

Assuming a power supply of 125 hp (745 W) per nozzle the water jet system is theoretically capable of scarifying rigid pavement to a depth of 2.5 in. (64 mm) at the rate of 0.58 ft^2/min (0.05 m^2/min)(12), considerably faster than the Klarcrete concrete cutter.

As a means of cost evaluation of the precast patches installed on Va-44, 22 cast-in-place patches with a total surface area of 101.5 ft^2 (9.4 m^2) were

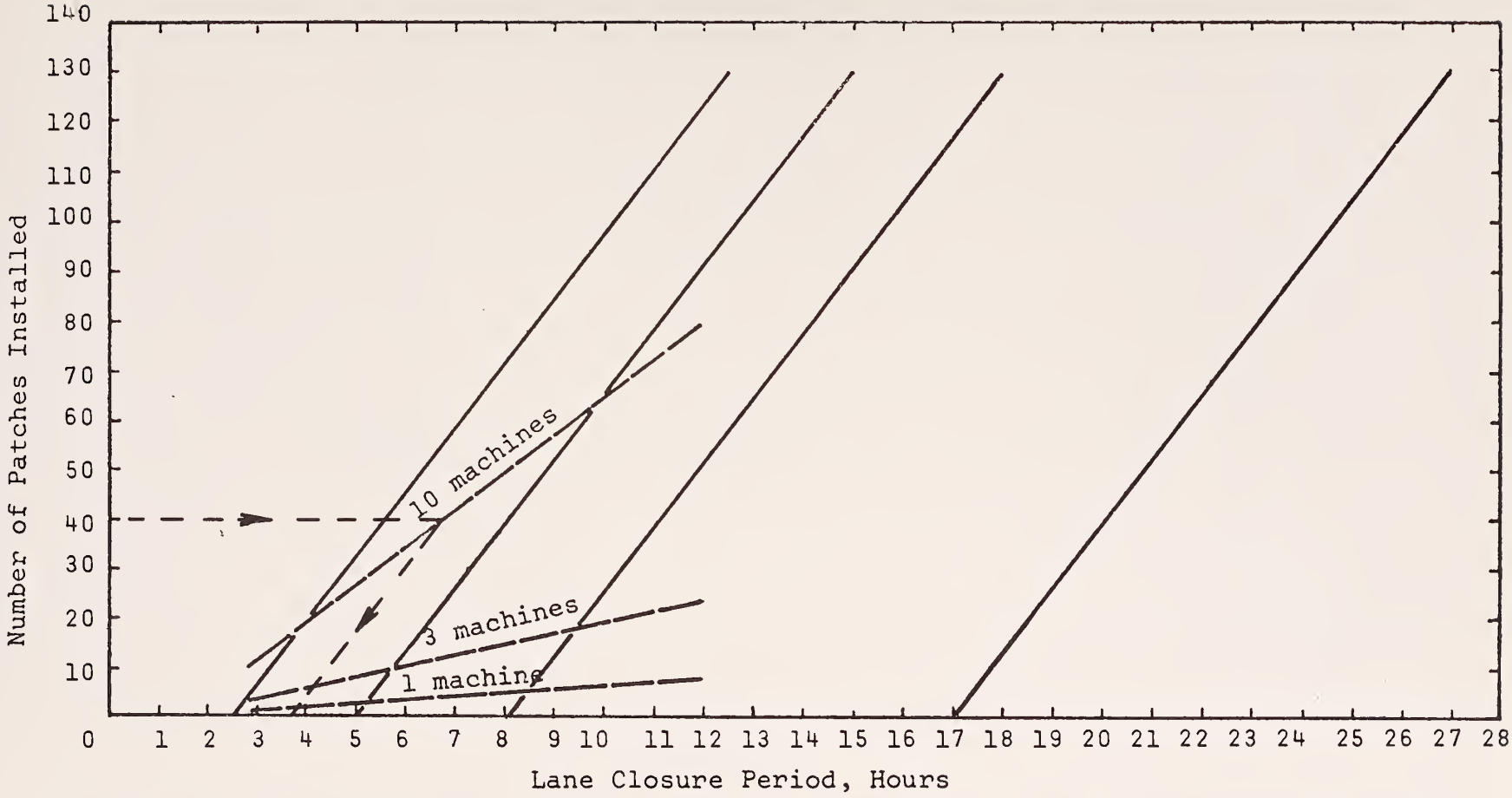


Figure 8-2. Production chart for precast patches installed using Klarcrete machine(s).

From Reference (11).

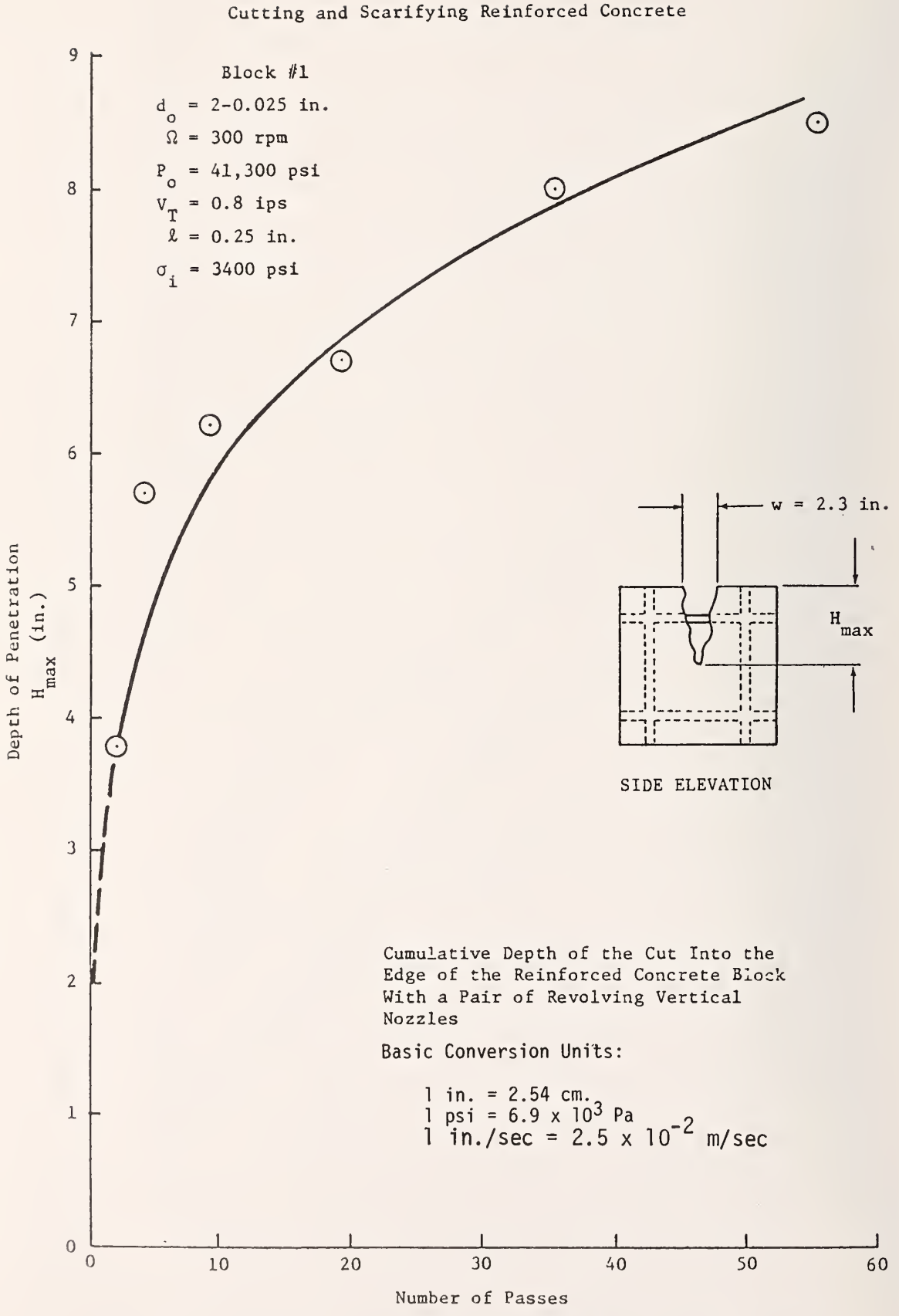


Figure 8-3. Cutting and Scarifying Reinforced Concrete

From Reference (12).

installed in the eastbound lane. The results of this evaluation showed the total unit cost using the precast technique was $29.80/ft^2$ ($320.00/m^2$) using the cast-in-place method.

A survey of the 90 patches (13) was conducted in July, 1976, two years following their installation (Table 8-5). In general all the patches have performed well. Hairline cracks were detected in 15 of the precast and 2 of the cast-in-place patches, however all have remained in place.

8.3 Full Depth Precast Slabs

Many types of distress in rigid pavements are manifested in failure of the pavement through its entire depth. Typical of these types of distress are blow up/compression failure, corner cracking, restraint cracking, transverse cracking, faulting and joint related distress. Appendix A summarizes the results of a survey of state transportation agencies. Tables A-13 and A-14 indicate the most prevalent and second most prevalent distress types. Satisfactory rehabilitation of these types of failures requires full depth and often full lane width removal of the distressed pavement to the extent that not only is structurally sound concrete exposed adjacent to the repair but the size of the repair must be adequate to provide a stable riding surface. Load transfer capability and allowances for volumetric variations of the pavement due to temperature differentials must also be adequately provided.

A typical rehabilitation technique for localized distress in rigid pavement would consist of removing the concrete by jackhammer or pavement breaker, leveling the disturbed base material, placing new concrete using rapid setting or high early strength cement and curing the concrete. Depending on the extent of the repair and the type of concrete used, the pavement is closed to traffic from 6 hrs to several days. Figure 8-4 shows a typical distribution of construction time. Traffic disruption of 6 hrs or more can be very expensive in terms of user cost on high traffic volume highways. Also the probability of accidents involving motorists and/or workmen increase with time of roadway occupancy.

An alternative rehabilitation technique that has been shown to be effective in not only providing a satisfactory repair but also substantially decreasing the time of traffic disruption is using full depth precast concrete paving

Table 8-5. Evaluation of Durability of Precast Concrete Patching After A Two-Year Time Span (July 1974 - July 1976)

PATCH NUMBER	SIZE OF PATCH WIDTH in.	m	LENGTH in.	m	TYPE OF PATCH	LANE	SOUNDING	APPEARANCE OF PATCH
1	12	.30	121.5	3.1	Hyd. Pressed[a] and Wirand[b]	Traffic	Good	No chips or cracks
2	24.5	.62	36	.91	Hyd. Pressed	Traffic	Good	No chips or cracks
3	25	.63	36	.90	Hyd. Pressed	Traffic	Good	No chips or cracks
4	18.5	.47	25	.63	Hyd. Pressed	Traffic	Good	No chips or cracks
5	14	.36	24.5	.62	Hyd. Pressed	Traffic	Good	No chips or cracks
6	13	.33	24	.61	Wirsod	Traffic	Good	No chips or cracks
7	25	.63	36	.90	Hyd. Pressed	Traffic	Good	Crack without movement upper left corner away from shoulder
8	12.5	.32	22	.56	Hyd. Pressed	Traffic	Good	No chips or cracks
9	13	.33	22.5	.58	Hyd. Pressed	Traffic	Good	No chips or cracks
10	12.5	.32	22.5	.58	Wirsod	Traffic	Good	No chips or cracks
11	23.5	.60	35.5	.90	Hyd. Pressed	Traffic	Unsound	Crack - lower left-hand corner; Crack - upper left-hand corner down to right corner
12	36	.91	47	1.2	Hyd. Pressed	Traffic	Unsound	Crack - left hand side halfway down toward shoulder; Unsound - left hand corner away from shoulder
13	13.5	.35	24	.61	Hyd. Pressed	Passing	Good	No chips or cracks
14	23	.58	25	.63	Hyd. Pressed	Passing	Unsound	Left hand corner hollow and cracked
15	22	.56	23.5	.60	Hyd. Pressed	Passing	Good	Hairline cracks
16	13.5	.35	25	.63	Hyd. Pressed	Passing	Good	No chips or cracks
17	23.5	.60	25	.63	Hyd. Pressed	Passing	Good	No chips or cracks
18	13	.33	22	.56	Hyd. Pressed	Passing	Good	No chips or cracks
19	23	.58	25	.64	Hyd. Pressed	Passing	Good	Hairline cracks
20	25	.64	35	.89	Hyd. Pressed	Passing	Unsound	Right-hand corner hollow, otherwise good
21	13.5	.35	23	.58	Hyd. Pressed	Passing	Unsound	Left-hand side, ½ of patch unsound
22	13	.33	22.5	.58	Hyd. Pressed	Passing	Unsound	Lower right-hand corner unsound
23	25	.64	36	.91	Hyd. Pressed	Passing	Good	No chips or cracks
24	18	.46	48.5	1.2	Hyd. Pressed	Passing	Good	No chips or cracks
25	18	.46	24	.61	Hyd. Pressed	Passing	Good	No chips or cracks
26	14.5	.37	25.5	.65	Wirsnd	Passing	Good	No chips or cracks
27	12.5	.31	13	.33	Wirsnd	Passing	Good	No chips or cracks
28	13	.33	87	2.2	Wirsnd and Polymer Injected[c]	Passing	Good	Cracked and chipped in polymer injected patch (middle patch)
29	18.5	.47	24	.61	Hyd. Pressed	Passing	Good	No chips or cracks
30	18	.46	25	.64	Hyd. Pressed	Passing	Good	No chips or cracks
31	25	.64	36	.91	Hyd. Pressed	Passing	Good	No chips or cracks
32	18	.46	24.5	.62	Hyd. Pressed	Passing	Good	No chips or cracks
33	13.5	.35	24.5	.62	Polymer Injected	Passing	Good	No chips or cracks
34	18	.46	24.5	.62	Hyd. Pressed	Passing	Good	No chips or cracks
35	19	.48	25	.64	Embeco[d]	Passing	Good	No chips or cracks
36	25.5	.66	50	1.3	Embeco	Passing	Unsound	Top 3/4's of patch adjacent to joint unsound
37	19	.48	37.5	.92	Hyd. Pressed	Traffic	Good	No chips or cracks
38	18.5	.47	24.5	.62	Hyd. Pressed	Traffic	Good	Crack - right 1/3 of patch
39	24	.61	36.5	.92	Hyd. Pressed	Traffic	Good	No chips or cracks
40	25	.64	50	1.3	Hyd. Pressed	Traffic	Good	No chips or cracks
41	25.5	.65	49.5	1.3	Wirsnd	Traffic	Good	No chips or cracks
42	25	.64	36	.91	Hyd. Pressed	Traffic	Good	Hairline cracks
43	14	.36	25	.64	Polymer Injected	Traffic	Good	No chips or cracks
44	25	.64	36	.91	Hyd. Pressed	Traffic	Good	Hairline cracks
45	18	.46	24.5	.62	Hyd. Pressed	Accel.	Good	Hairline cracks

Table 8-5. Evaluation of Durability of Precast Concrete Patching After A Two-Year Time Span (July 1974 - July 1976) (Continued)

PATCH NUMBER	SIZE OF PATCH WIDTH in.	m	LENGTH in.	m	TYPE OF PATCH	LANE	SOUNDING	APPEARANCE OF PATCH
46	25.5	.65	36	.91	Hyd. Pressed	Traffic	Good	Hairline cracks
47	23.5	.60	25.5	.65	Hyd. Pressed	Traffic	Good	No chips or cracks
48	25	.64	36	.91	Hyd. Pressed	Traffic	Good	No chips or cracks
9	22.5	.57	25.5	.65	Hyd. Pressed	Traffic	Good	No chips or cracks
50	18	.46	24	.61	Hyd. Pressed	Traffic	Good	Hairline cracks
51	19	.48	25	.64	Hyd. Pressed	Traffic	Good	No chips or cracks
52	24	.61	26	.66	Hyd. Pressed	Traffic	Good	Hairline cracks
53	24	.61	26	.66	Hyd. Pressed	Traffic	Good	No chips or cracks
54	25.5	.65	72	1.8	Embeco	Accel.	Unsound	Center portion of patch unsound
55	24	.61	26	.66	Embeco	Traffic	Good	No chips or cracks
56	19.5	.49	23.5	.60	Embeco	Traffic	Good	No chips or cracks
57	37	.94	50	1.3	Embeco	Traffic	Good	No chips or cracks
58	18	.46	25	.64	Embeco	Traffic	Good	No chips or cracks
59	18	.46	25	.64	Embeco	Traffic	Good	No chips or cracks
60	13.5	.35	18	.46	Embeco	Traffic	Good	Chipped upper right hand corner near joint
61	19	.48	24.5	.62	Embeco	Traffic	Good	No chips or cracks
62	13.5	.35	21	.53	Embeco	Traffic	Good	No chips or cracks
63	13.5	.35	18.5	.47	Embeco	Traffic	Good	No chips or cracks
64	13	.33	18	.46	Embeco	Traffic	Good	No chips or cracks
65	13	.33	19	.48	Embeco	Traffic	Good	No chips or cracks
66	13.5	.34	31.5	.79	Embeco	Traffic	Good	No chips or cracks
67	13.5	.34	19	.48	Embeco	Traffic	Good	No chips or cracks
68	25	.64	36.5	.92	Embeco	Passing	Good	No chips or cracks
69	37	.94	76.5	1.9	Embeco	Passing	Good	No chips or cracks
70	14.5	.37	27	.69	Embeco	Passing	Good	No chips or cracks
71	26	.66	49.5	1.3	Embeco	Passing	Good	No chips or cracks
72	11.5	.29	24.5	.62	Embeco	Passing	Good	No chips or cracks
73	19	.48	27	.69	Embeco	Passing	Good	No chips or cracks
74	25	.64	28	.71	Hyd. Pressed	Passing	Good	No chips or cracks
75	25	.64	48.5	1.2	Hyd. Pressed	Passing	Unsound	Upper right hand corner of patch unsound
76	22.5	.57	25	.64	Hyd. Pressed	Traffic	Good	No chips or cracks
77	19	.48	25.5	.65	Hyd. Pressed	Traffic	Good	No chips or cracks
78	24.5	.63	27	.69	Hyd. Pressed	Traffic	Good	Vertical crack in middle of patch
79	14.5	.37	25	.64	Wirand	Traffic	Good	No chips or cracks
80	24	.61	25	.64	Wirand	Traffic	Unsound	Upper right corner unsound
81	26	.66	36	.91	Hyd. Pressed	Passing	Good	No chips or cracks
82	25	.64	50	1.3	Wirand	Passing	Good	No chips or cracks
83	25	.64	36.5	.93	Hyd. Pressed	Passing	Good	No chips or cracks
84	25	.64	50.5	1.3	Wirand	Passing	Good	No chips or cracks
85	19.5	.50	24	.61	Hyd. Pressed	Passing	Good	No chips or cracks
86	13	.33	25	.64	Wirand	Passing	Good	No chips or cracks
87	18	.46	24.5	.63	Hyd. Pressed	Passing	Good	No chips or cracks
88	18	.46	24.5	.63	Hyd. Pressed	Passing	Good	No chips or cracks
89	24	.61	25	.64	Wirand	Passing	Good	No chips or cracks
90	24	.61	25	.64	Wirand	Passing	Good	No chips or cracks

(a) Hydraulically pressed.
(b) Wirand- precast patches with metal wings incorporated.
(c) Polymer Injected- precast patches impregnated with a 9 to 1 mixture of methyl methacaylate and trimethylolpropane and 1% by weight of Vazo 52 solution. Treatments to determine if salt action is reduced.
(d) Embeco- concrete containing iron filings to reduce shrinkage.

From Reference 13.

units. Both normally reinforced and prestressed concrete units have been used in a variety of sizes ranging from 4 ft x 12 ft x 6.75 in. thick (1.22 m x 3.66 m x 16 cm) to 13 ft x 30 ft x 6.75 in. thick (3.96 m x 9.14 m x 16 cm).

An experimental project conducted in 1973 by maintenance personnel in Florida demonstrated the feasibility of using precast elements (14). The project was located on an urban section of the Interstate System constructed as an unreinforced jointed pavement in the mid 1950's. The distress was manifested as cracks in the slabs and had been either ignored or temporarily repaired by applying a leveling course of asphaltic concrete. A permanent repair was needed, but due to the traffic volume, long term lane closures were unacceptable. The solution was the development of a repair technique using precast concrete slabs as the replacement of the weaker existing slabs. Each broken slab was accurately measured and its replacement was cast in a "custom fit" manner since the joint spacing and angle of the joints relative to the pavement centerline were not consistent. The slabs were cast 8 in. (200 mm) thick using 3000 psi (21,000 kPa) concrete, consolidated by a vibrating screed and given a light broom finish. Reinforcing was designed to compensate for handling and placement stress and consisted of two mats of Number 5 bars (16 mm) spaced 6 in. (150 mm) on center longitudinally and 15 in. (380 mm) on center transversely.

Removal of the broken slab was accomplished by sawing full depth along the joints surrounding the slab, breaking the slab into pieces suitable for handling with a crane and drop ball and loading the debris for disposal. The subbase material was then leveled and machine compacted to a grade that would allow the surface of the new panel to be 1/2 in. (12 mm) below the adjacent pavement. The precast slab was then placed on the subbase and mudjacked to its final position. A pressed fiberboard was then placed in the joints surrounding the slabs and finally the joints were sealed with a hot rubberized asphalt sealant.

The repairs were made between the hours of 10:00 pm and 6:00 am when traffic volume was at a minimum. During this 8 hr period as many as four slabs or 106 yd^2 (89 m^2) of pavement were replaced including the time required to set up and remove traffic control devices.

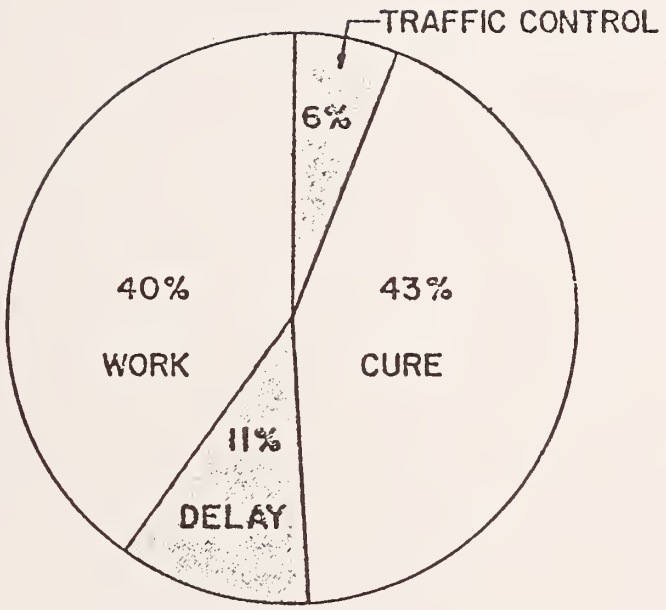


Figure 8-4. Average distribution of observed roadway occupancy time for concrete, cast-in-place, full depth repairs

From Reference (10).

The unit cost of pavement rehabilitation associated with this technique was reported as $35/yd^2$ ($42/m^2$) in 1972, compared to an estimated $40/yd^2$ ($48/m^2$) for cast-in-place (15).

As reported by Grimslay (14) several problems were encountered using this technique that, if resolved, could have a substantial influence on the cost. The major problems were lack of proper equipment and facilities and the lack of experienced personnel due to the infrequency of using the technique. Due to a change in the agency's maintenance strategy the precast technique is not currently being employed, however, it is still considered feasible and would likely be used if slab replacement is required in certain high traffic volume areas.

A June, 1977, inspection of the precast slabs revealed they were performing well. Very little faulting at the joints was observed. Several of the slabs had cracked but the reinforcing steel was providing load transfer across the cracks and preventing the crack from opening. The precast repairs appear to be capable of providing satisfactory service for several more years.

A similar project was completed in California in 1974 (17). Two precast paving slabs were installed in the Ventura Freeway as replacement of deteriorated pavement resulting from the incorporation of a bad load of concrete during construction. The slabs measured 11.4 ft x 12.5 ft x 0.67 ft thick (3.47 m x 3.81 m x 0.2 m) and 11.4 ft x 17.4 ft x 0.67 ft thick (3.47 m x 5.3 m x 0.2 m) and weighed 14,000 lb (6,350 kg) and 20,000 lb (9,720 kg) respectively. Reinforcement consisted of two mats; the top one containing Number 4 (12.7 mm) bars at 18 in. (460 mm) on center and the bottom mat Number 4 (12.7 mm) bars at 12 in. (300 mm) on center. A concrete mix containing 7.8 sacks of cement per yd^3 ($250 kg/m^3$) and 0.4% Zeecon[a] was used to cast the panel yielding a flexural strength of 652 psi (4,500 kPa) in six days.

Removal of the distressed pavement was accomplished by breaking the concrete with an Arrow Hydrohammer into sizes suitable for loading with a Catapiller 950 rubber tired loader. Prior to placing the precast panels a 5-6 in. (130 mm- 150 mm) slump, 7 sack, sand-cement grout was placed in the repair area to provide uniform support for the panels. The panels were bedded into the grout which had been struck off approximately one hour before to allow curing

[a]Zeecon is a lignin water reducing admixture manufactured by Crown Zellerbach Corp., 1 Bush St., San Francisco, California 94119.

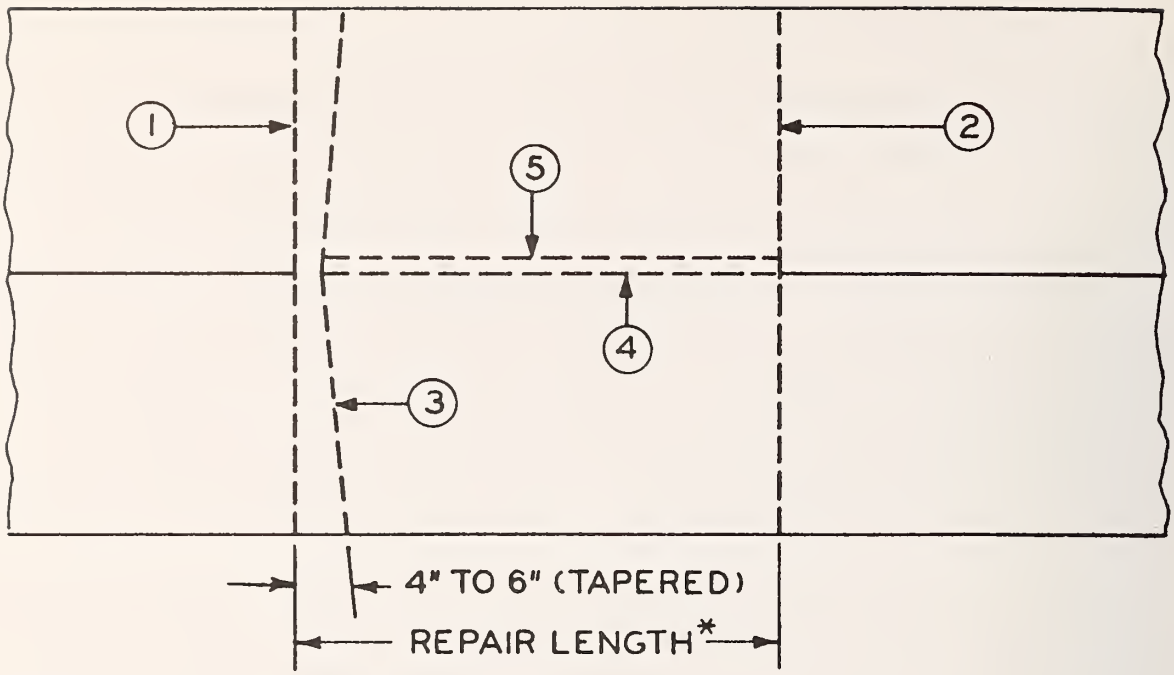
of the grout. Grout was then placed in the crack around the repair and the pavement was opened to traffic.

A total of 37.6 yd^2 (31.4 m^2) of pavement was replaced in 7 hrs and 15 min. including delays and time required for traffic control at a 1974 unit cost of \$78/yd^2 (\$93/m^2). The cost of the repair was relatively high but the inconvenience to motorists and the exposure of workmen to traffic was reduced considerably below the normal 7 to 10 days required for more conventional rehabilitation techniques. In his report of this project, Foor (17) suggested that improved procedures derived from studies of this technique could reduce the cost and time requirements.

A similar technique has been developed by the Michigan Department of Highways and Transportation from an experimental project conducted in 1969 (6). The technique has evolved into standard maintenance practice using full lane width slabs 6, 8, 10, and 12 ft (1.8, 2.4, 3.0, and 3.7 m) long in the replacement of spalled joints. The thickness of the slabs varied depending on the thickness of the pavement being repaired. The panel thickness was typically 1 in. (25 mm) less than the existing pavement to provide space for the bedding grout.

Pavement removal was accomplished by full depth sawing of the pavement around the spalled joints and making an additional cut across one end creating a slightly tapered section of pavement approximately 4 in. (100 mm) wide. The taper facilities removal of the narrow section which, when removed, eliminates potential difficulty in removal of the distressed concrete resulting from thermal expansion of the pavement. Figure 8-5 illustrates the concrete cutting procedure. Connecting holes are then drilled in the pavement to be removed and it is lifted from its location and loaded onto a truck for disposal. This method was developed to minimize disturbance of the subbase material thus providing support for the repair slab exactly the same as that for the existing pavement.

The length of lane removed is determined by the extent of spalling plus a minimum of 4 in. (100 mm) allowing for contraction or expansion joints on each end of the repair. Contraction joints are constructed by filling the space between the existing pavement and precast slab with a sand-cement mortar to



* Repair length as specified in the log of work or as directed by the Engineer. (For precast slabs, length of repair equals length of precast slab plus 4 in.)

(1) (2) These sawcuts shall be perpendicular to the edge of the roadway. If the repair is on a curve the cuts shall be perpendicular to a straight line extended between the limits of the repair at the edge of the roadway.

(3) This cut is made to facilitate opening a trench across the slab to relieve compression in the pavement prior to lifting out the failed areas.

(4) This cut is made between lanes, between a lane and a curb, and between a lane and a ramp.

(5) This cut is made for the reasons given in 3 except for facilitating pressure relief in the transverse direction. The cut can be omitted if there are no adjacent lanes, curbs or ramps.

Additional sawcuts may be made inside the repair limits to reduce weight and size of concrete pieces to be lifted out.

The specified distance between sawcuts (1) and (2) shall be held within a tolerance of -0 + 1/2 in. in 12 ft.

All saw cuts to be full-depth.

Figure 8-5. Sawing Diagram

1 in. = 2.54 cm
1 ft = 0.305 m

From Reference (18).

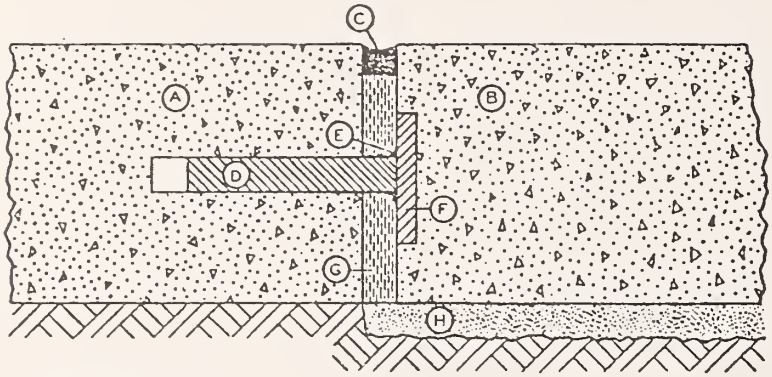
within 1 1/2 in. (38 mm) of the surface and, after curing the mortar, sealing the joint with a hot-poured rubber-asphalt sealant. Expansion joints are constructed by installing a premolded bituminous filler in the space provided and sealing the joint.

Load transfer devices were installed in five expansion joints in the experimental projects (18). The joints were constructed by drilling eleven 1 1/4 in. (32 mm) holes 9 in. (230 mm) deep in the sawed vertical face of the existing pavement and inserting 1 3/16 in. (30 mm) diameter 9 in. (230 mm) long dowels in the holes prior to placing the precast slab. Following placement and alignment of the slab the dowels were pulled out of the holes to make contact with a 3/8 in. (10 mm) steel plate cast into the edge of the precast slab. The dowels were welded to the plate, a slatted bituminous filler board was placed in the joint and the joint was sealed. Typical doweled and undoweled joints are illustrated in Figure 8-6.

Results of the experiment showed only minor difference in the faulting between doweled and undoweled joints. After four years of service the doweled joints had faulted a maximum of 0.12 in. (3 mm) while the undoweled joints had faulted a maximum of 0.32 in. (8 mm). Installation of the load transfer devices required an additional 1 1/4 hours of construction time over the undoweled precast installation. As a result of this experiment, the installation of load transfer devices was not considered worthwhile by the Michigan Personnel.

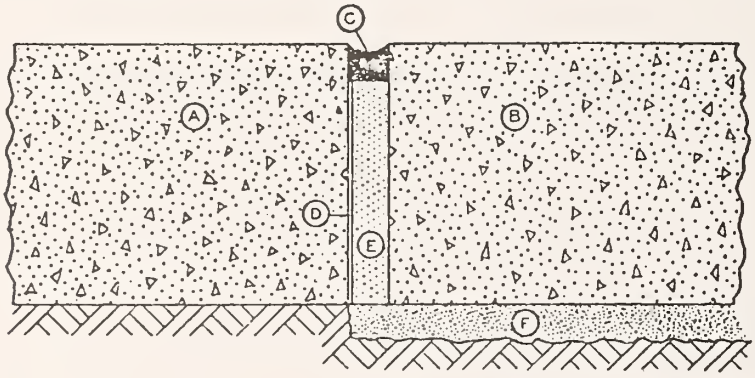
Performance of the precast joint repairs relative to conventional cast-in-place repairs were also studied in the experimental project. The extent of the experimental project is shown in Table 8-6. Performance was evaluated by measuring joint movement, joint faulting, slab rocking and deflection under load. In addition to these measurements, periodic visual surveys were made to monitor repair performance.

Results of the joint movement measurements are given in Figures 8-7 and 8-8. The measurements show that in both the precast and cast-in-place sections the joints have, in general, continuously closed. Visual inspection of the repair in July, 1977 (19), showed that the trend has continued to the extent that many of the joints are completely closed. The reason for this was observed to be the presence of transverse cracks in the pavement that have allowed the intrusion of incompressable material during periods of cold weather when the pavement was



(A) EXISTING SLAB
(B) PRECAST SLAB
(C) LIQUID JOINT SEALER
(D) STEEL DOWEL
(E) FIELD WELD
(F) STEEL PLATE
(G) BITUMINOUS JOINT FILLER
(H) SAND-CEMENT MORTAR

DOWELED JOINT



(A) EXISTING SLAB
(B) PRECAST SLAB
(C) LIQUID JOINT SEALER
(D) BOND BREAKER (ONE END ONLY)
(E) EPOXY MORTAR
(F) SAND-CEMENT MORTAR

NON-DOWELED JOINT

Figure 8-6. Details for Doweled and Undoweled Joints

From Reference (18).

TABLE 8-6.
SUMMARY OF JOINT REPAIR DETAILS

	Repair Type	No. of Repairs	Repair Length, ft	Base Support	Joint Type	Filler Material	Nominal Joint Width, in.
I 96 Westbound Roadway (1972)	Precast	1	10	Mortar	Contraction	Mortar[1]	0
	Precast	1	10	Sand	Contraction	Mortar	0
	Precast	1	10	Mortar	Expansion	Bit. Filler[1]	1-1/2
	Precast	1	10	Sand	Expansion	Bit. Filler	1-1/2
	Precast[2]	1	10	Mortar	Expansion	Bit. Filler	1-1/2
	Precast[2]	1	10	Sand	Expansion	Bit. Filler	1-1/2
	Cast-in-Place	1	10	Ex. Base	Contraction	None	0
	Cast-in-Place	1	10	Ex. Base	Expansion	Bit. Filler	1-1/2
	Cast-in-Place[2]	1	10	Ex. Base	Expansion	Bit. Filler	1-1/2
	Cast-in-Place[3]	1	10	Ex. Base	Expansion	Bit. Filler	1-1/2
I 96 Eastbound Roadway (1973)	Precast	2	10	Sand	Expansion	Polyethylene	2
	Precast	2	8	Sand	Expansion	Polyethylene	2
	Precast	2	6	Sand	Expansion	Polyethylene	2
	Cast-in-Place	1	14	Ex. Base	Expansion	Polyethylene	2
	Cast-in-Place	5	10	Ex. Base	Expansion	Polyethylene	2
	Cast-in-Place	2	8	Ex. Base	Expansion	Polyethylene	2
	Cast-in-Place	2	6	Ex. Base	Expansion	Polyethylene	2
	Cast-in-Place[3]	2	5	Ex. Base	Expansion	Polyethylene	2
	Cast-in-Place[3]	2	4	Ex. Base	Expansion	Polyethylene	2
I 69 Westbound Roadway (1974)	Cast-in-Place	1	12	Ex. Base	Expansion	Polyethylene	1-1/3
	Cast-in-Place	1	12	Ex. Base	Expansion	Bit. Filler	1
	Cast-in-Place	2	10	Ex. Base	Expansion	Polyethylene	1-1/3
	Cast-in-Place	2	8	Ex. Base	Expansion	Polyethylene	1-1/3
	Cast-in-Place	6	8	Ex. Base	Expansion	Bit. Filler	1
	Cast-in-Place	4	6	Ex. Base	Expansion	Polyethylene	1-1/3
	Cast-in-Place	4	6	Ex. Base	Expansion	Bit. Filler	1

1 Contraction joints, and expansion joints constructed with bituminous filler, were sealed with hot-poured rubber-asphalt sealant.
2 Load transfer provided by installation of steel dowels on 12-in. centers.
3 These repairs have one hook-bolted joint and one expansion joint. The expansion joint with the 1972 repair of this type is doweled.

1 in. = 2.54 cm

From Reference (9).

1 ft = 0.305 m

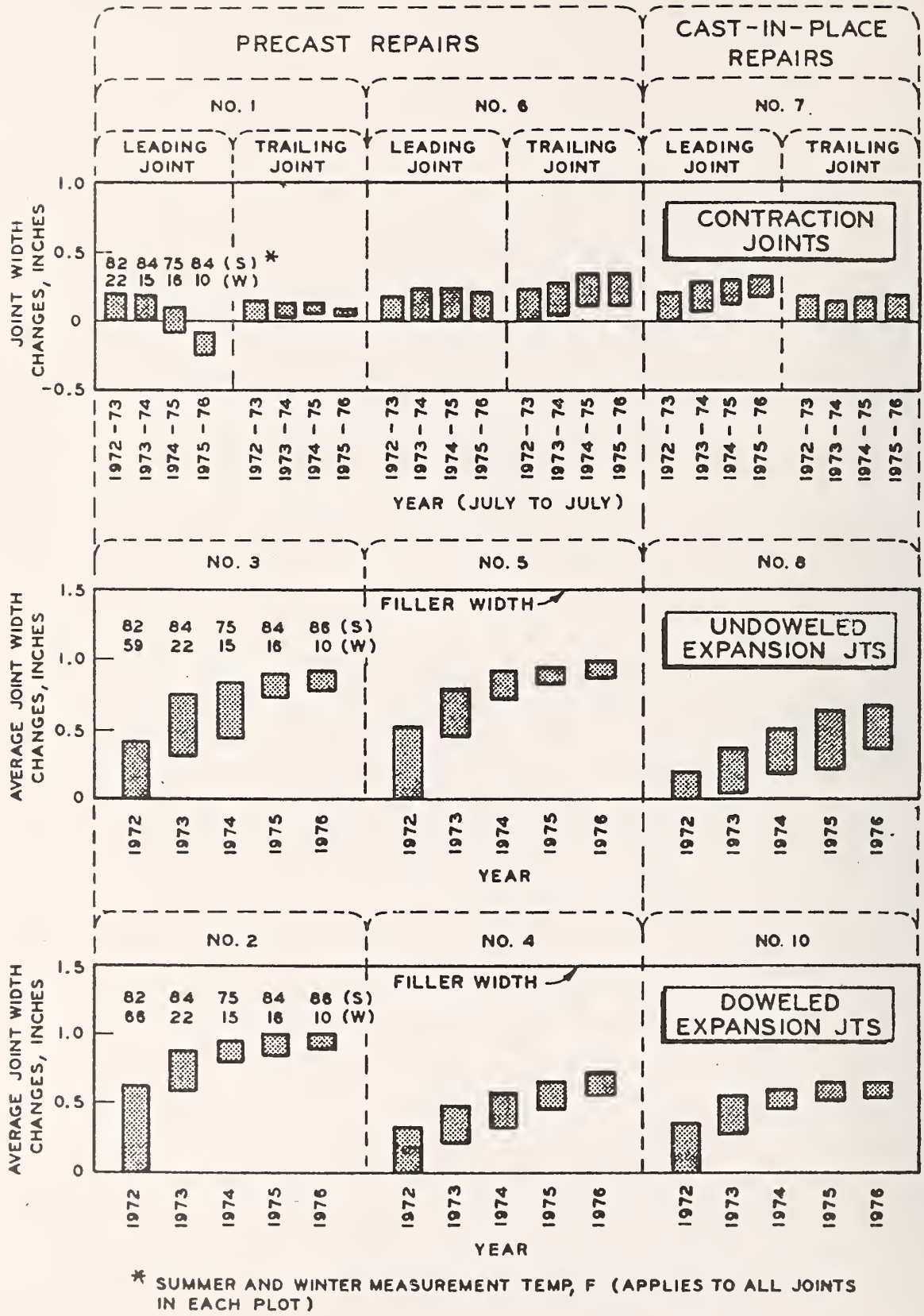


Figure 8-7. Average seasonal and yearly joint width changes at repairs installed on westbound I 96 in 1972.

1 in. = 2.54 cm

From Reference (9).

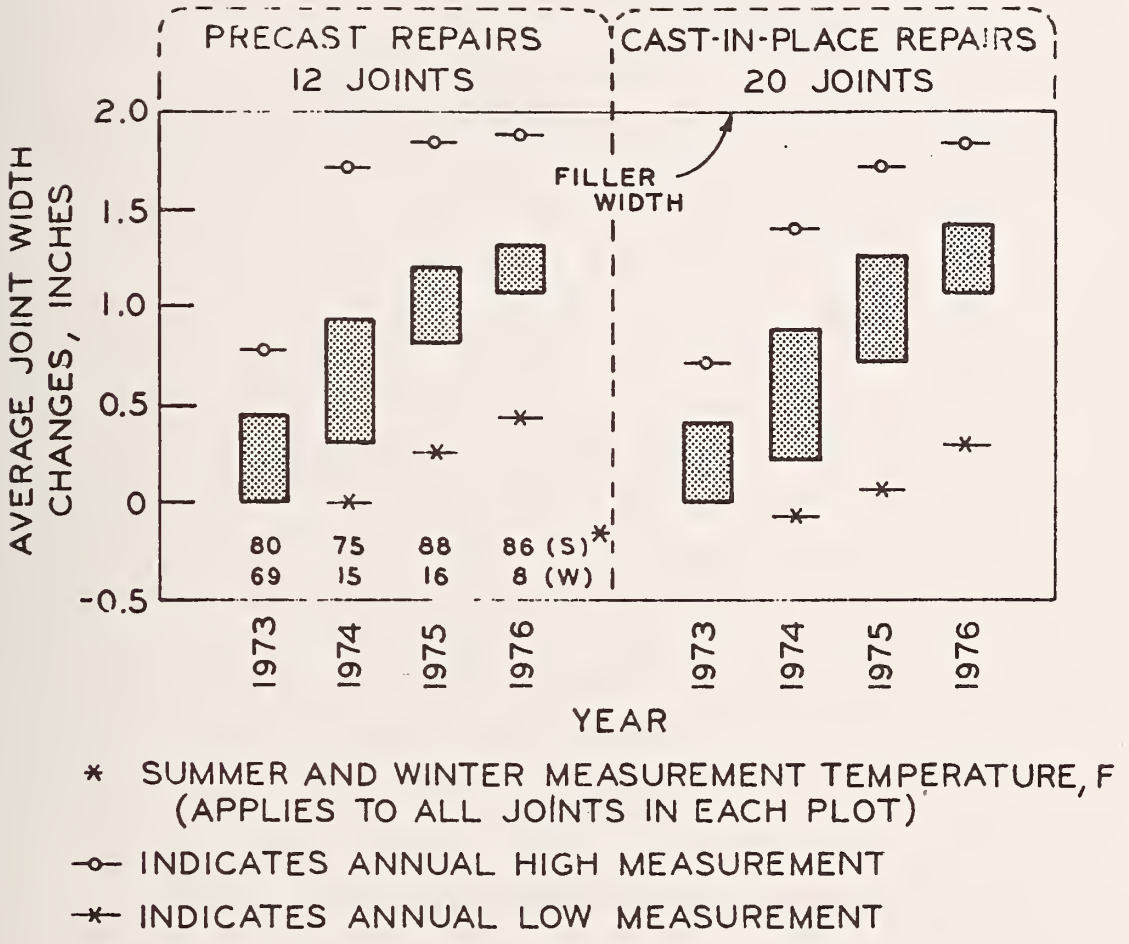


Figure 8-8. Average seasonal, yearly, and yearly high-low joint width changes at repairs installed on eastbound I 96 in 1973.

1 in. = 2.54 cm

From Reference (9).

contracted. When the pavement expands during periods of warmer weather the expansion is absorbed by the sealed joints rather than the transverse cracks. Several cracks were observed to be as much as 1 1/2 in. (40 mm) wide. This problem will probably result in spalling of the joints at some time in the future -- requiring further joint repair.

As pointed out earlier joint faulting studies showed a relatively small difference in initial faulting of the doweled or undoweled joints. In general, faulting was more severe at the trailing edge of the repair. Figures 8-9 and 8-10 show the results of the joint faulting measurements.

Slab rocking was determined by measuring vertical movement of the leading and trailing edge of the repair as an 18,000 lb (8165 kg) axle load moved over it. No significant difference was noted in the two types of repair for slabs 10 ft (3 m) long. However, precast slabs less than 10 ft (3 m) in length exhibited considerable movement. Figure 8-11 shows the results of these measurements.

Load deflection measurements were taken at a point 2 in. (50 mm) from the pavement edge and 2 in. (50 mm) on either side of the joint as an 18,000 lb (8165 kg) load moved over the pavement at a distance of 12 in. (30 cm) from the edge. Results of these measurements are shown in Figures 8-12 and 8-13.

The experimental project revealed that in general, precast slabs perform equally well as cast-in-place slabs when the repair was greater than 8 ft (2.4 m) in length. Undoweled precast slabs less than 8 ft (2.4 m) in length exhibited slightly more rocking and joint faulting than did the undoweled cast-in-place repairs. However, doweled precast and undoweled precast slabs greater than 8 ft (2.4 m) in length show performance comparable to the cast-in-place repair.

The project also showed that repairs using precast slabs could be completed in 1 1/2 hrs while cast-in-place repairs of the same size using rapid setting cement required lane closure times of at least 6 hrs.

Specifications for pavement repair using precast concrete slabs were developed by the State of Michigan Department of State Highways and the technique was implemented as a standard maintenance practice.

In 1972 approximately 2,450 yd^2 (2048 m^2) of precast repairs were installed. The average unit price was reported by Simonsen (18) as $51.50 per yd^2 (per 0.84 m^2) while the average

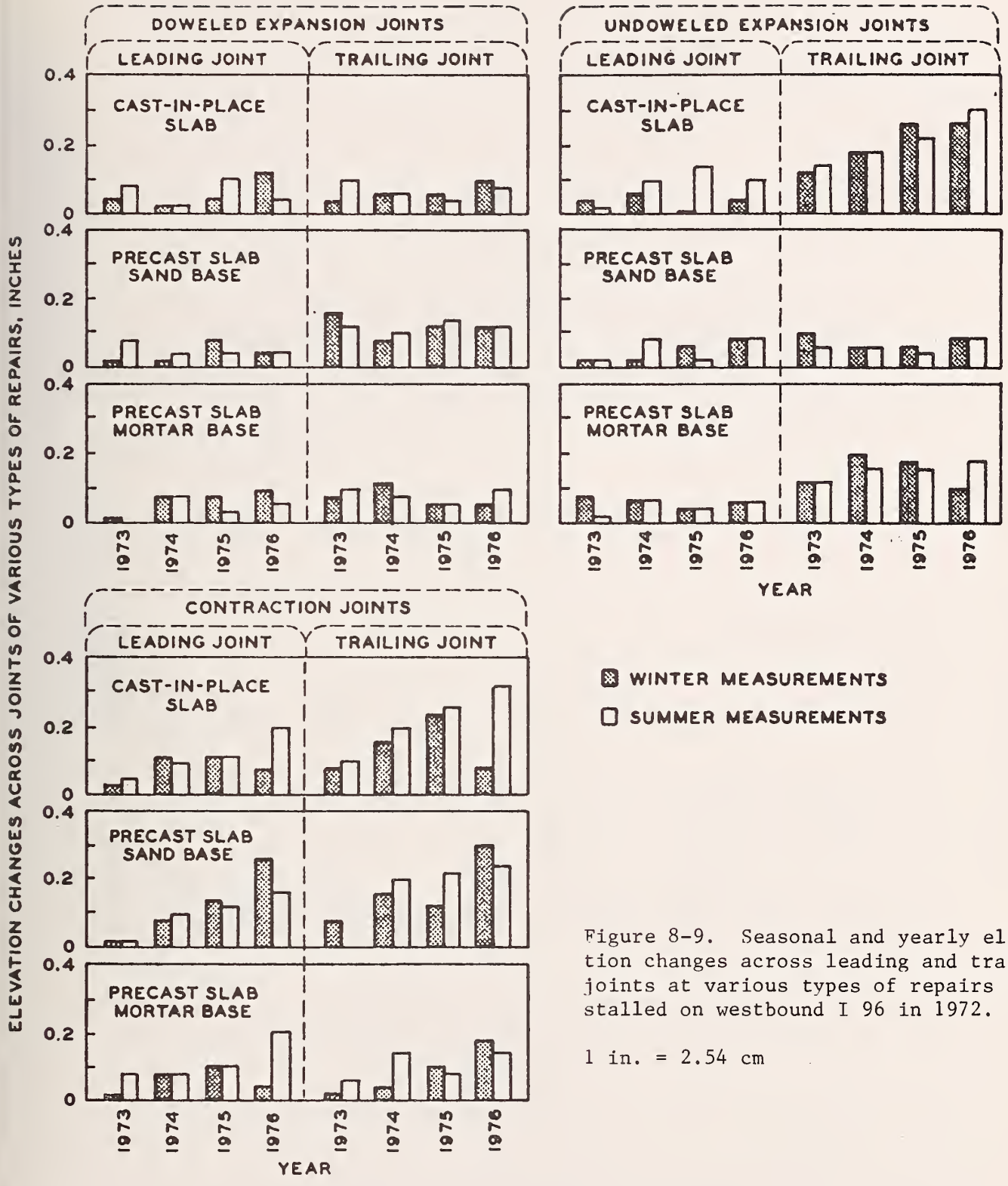


Figure 8-9. Seasonal and yearly elevation changes across leading and trailing joints at various types of repairs installed on westbound I 96 in 1972.

1 in. = 2.54 cm

From Reference (9).

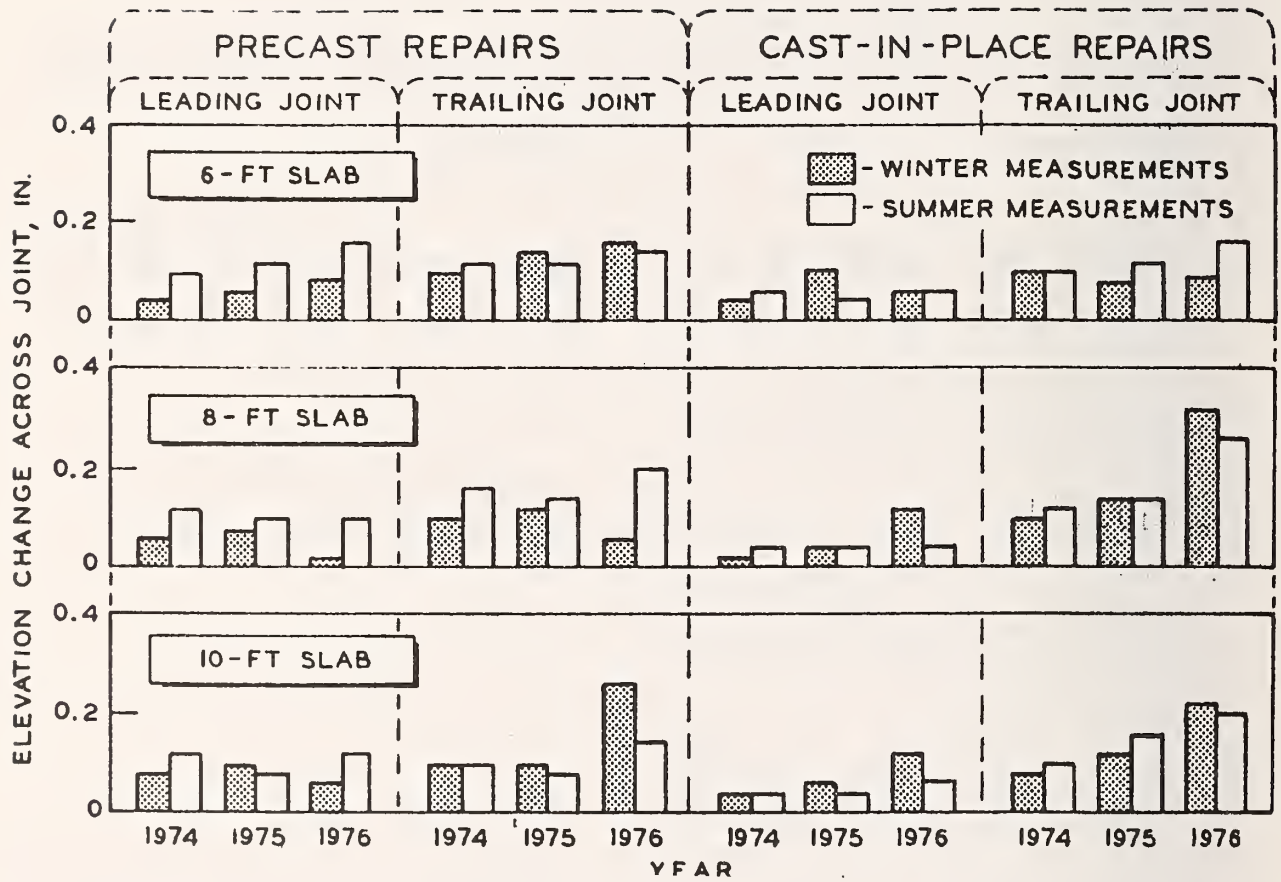


Figure 8-10. Average seasonal and yearly elevation changes across leading and trailing expansion joints at 6, 8, and 10-ft long precast and cast-in-place repairs installed on eastbound I 96 in 1973.

1 in. = 2.54 cm

1 ft = 0.305 m

From Reference (9).

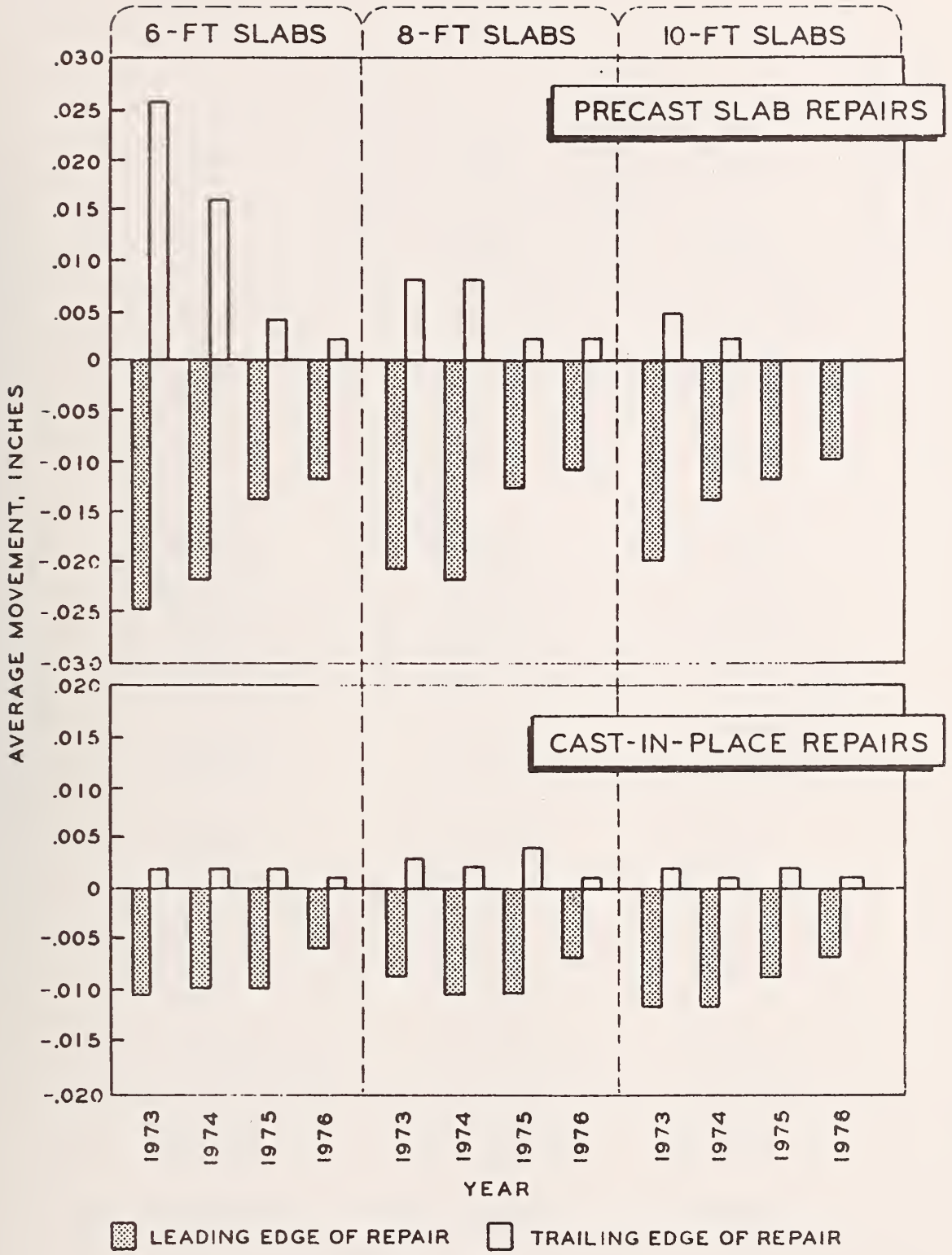


Figure 8-11. Rocking of repair slabs on eastbound I 96 precast repairs (top) and cast-in-place repairs (bottom).

1 in. = 2.54 cm

1 ft = 0.305 m

From Reference (9).

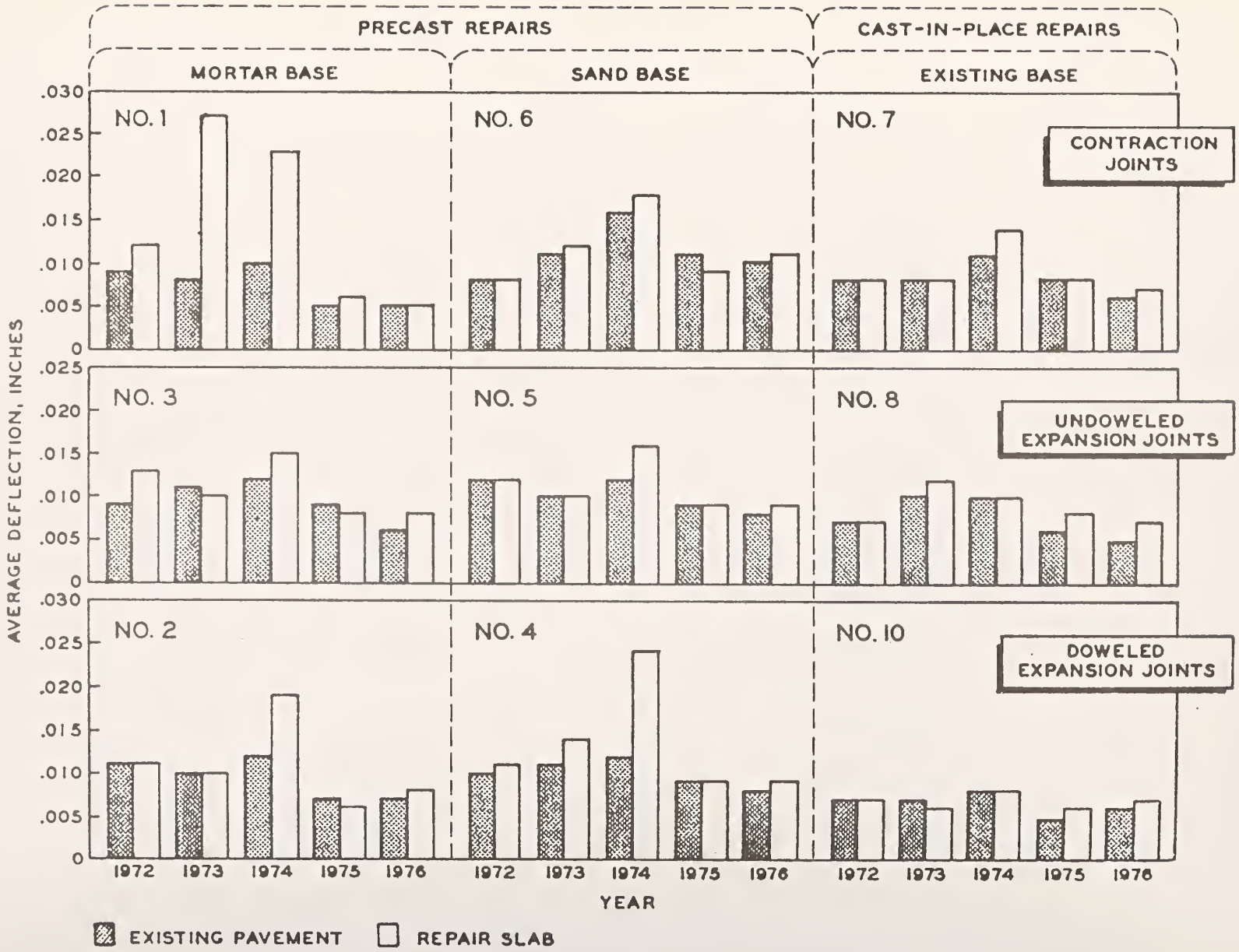


Figure 8-12. Daytime load deflection versus time at the leading joint of precast and cast-in-place repairs on westbound I 96. 1 in. = 2.54 cm

From Reference (9).

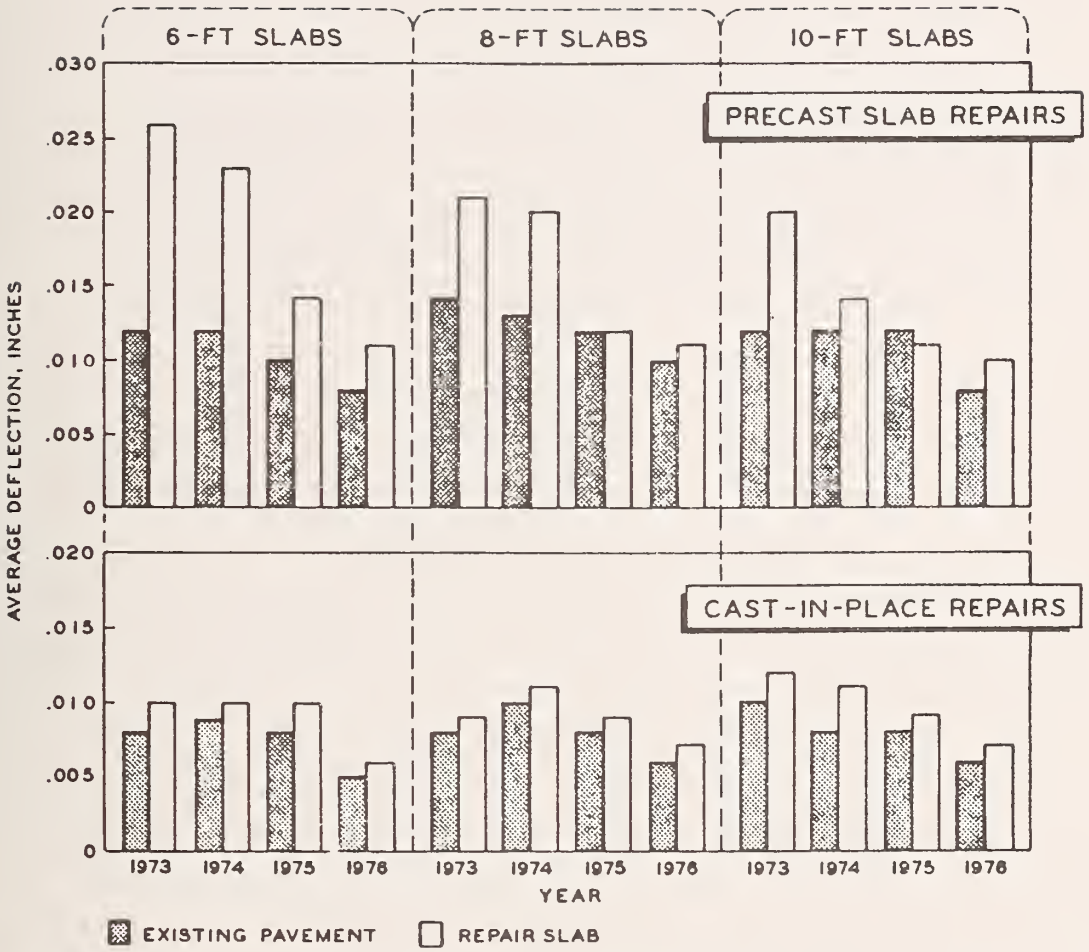


Figure 8-13. Average daytime load deflection versus time of the leading joint of precast and cast-in-place repairs on eastbound I 96.

1 in. = 2.54 cm

1 ft = 0.305 m

From Reference (9).

unit price for installing approximately 7,500 yd^2 (6270 m^2) of cast-in-place repairs was \$46.00/yd^2 (\$55.00/m^2).

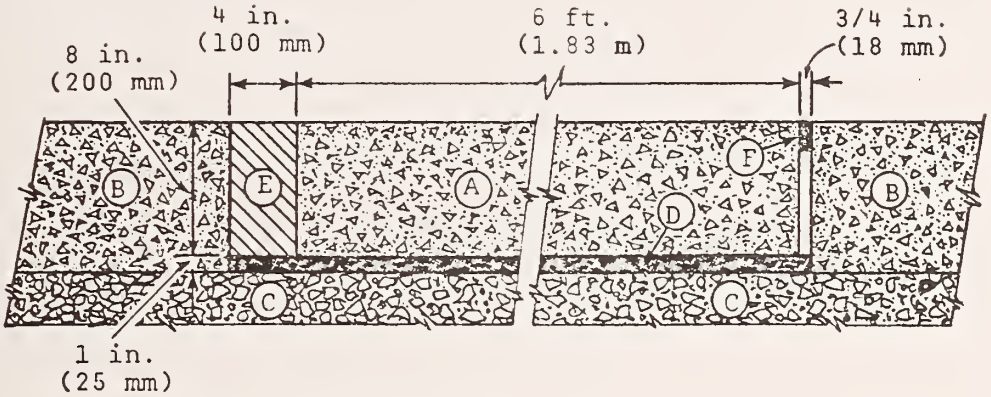
8.4 POTENTIAL PRECAST REHABILITATION TECHNIQUES

The previous discussion describes repair techniques that have been developed and implemented by various maintenance agencies. The techniques have all shown to be feasible when applied under the conditions for which they were designed. From the experience gained through these projects new techniques could possibly be developed that would be feasible under a wider range of conditions. The following is a discussion of several potentially feasible techniques.

8.4.1 Precast Joint Assembly. This technique is discussed in Chapter 6. The application of this technique could be suitable for replacement of the precast techniques used to repair spalled joints as reported by Simonsen (6), (10), (18). The obvious advantages this new technique would offer over the cast-in-place method and the existing precast method is the load transfer capability of the single joint after the repair is completed rather than the two undoweled joints present with the methods currently being used. The minor faulting that typically occurs with precast slabs would be eliminated by bonding the precast slab to the existing pavement with an epoxy grout.

8.4.2 Precast Panel Unit. The greatest problem with existing rehabilitation techniques using precast panels is the lack of adequate load transfer across joints often resulting in a slight amount of faulting (19). The typical precast slab technique includes an expansion joint at either end of the slab. Figure 8-14 illustrates the transverse joints of the precast technique developed by the Virginia Highway and Transportation Research Council (20). The development of an alternative technique to include physical attachment of the existing and precast pavements is complicated by the many possible conditions under which rehabilitation is required.

Three possible transverse joint details are herein presented with the anticipation that one will be applicable to each or both ends of a precast panel depending on the rehabilitation conditions. Joint Detail A in Figure 8-15 provides a means of preventing typical precast panel joint faulting as observed in Michigan (19), when installed in an unreinforced pavement. This pavement will be cut with a concrete saw at an appropriate angle such that when



A — Precast Slab
B — Existing Slabs
C — Subbase
D — Bedding Mortar
E — Stress Relief Material
F — Preformed Neoprene Sealant

Figure 8-14. Elements of a typical precast replacement installation (longitudinal section).

From Reference (20).

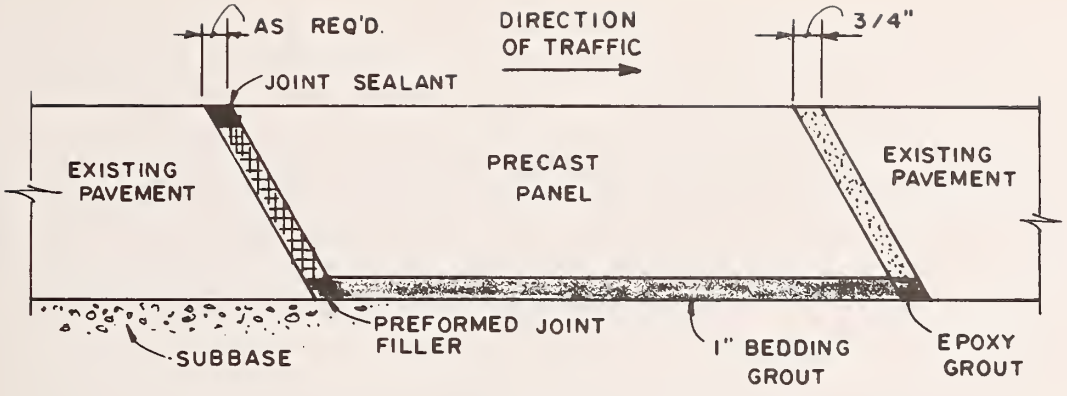
the following edge joint is filled with a nonshrinking epoxy grout, upward movement resulting in faulting will be prevented. An expansion joint can be provided as shown at the leading edge joint or this joint could also be bonded to the existing pavement if the particular installation did not require temperature compensating capabilities.

Joint Detail B of Figure 8-15 illustrates a tied joint for use in reinforced pavement. The pavement will be cut using a technique such as the high pressure water jet system developed by Flow Research (12) whereby the existing reinforcing steel will not be damaged. The water jet will be used to trench the concrete along the reinforcing bar so that when the bar is cut a tie bar of adequate length for welding to the continuous plate will be provided. Reinforcing bars will be stubbed out of the precast panel and also welded to the two continuous plates placed as shown in Detail B. The size of the weld plates will be determined by the space between the stubbed out bars. Some bending of the bars should be possible to provide a uniform space for the plates and to bring the bars into contact with the plates for welding. The area surrounding the connection will then be filled with a non-shrinking cement grout.

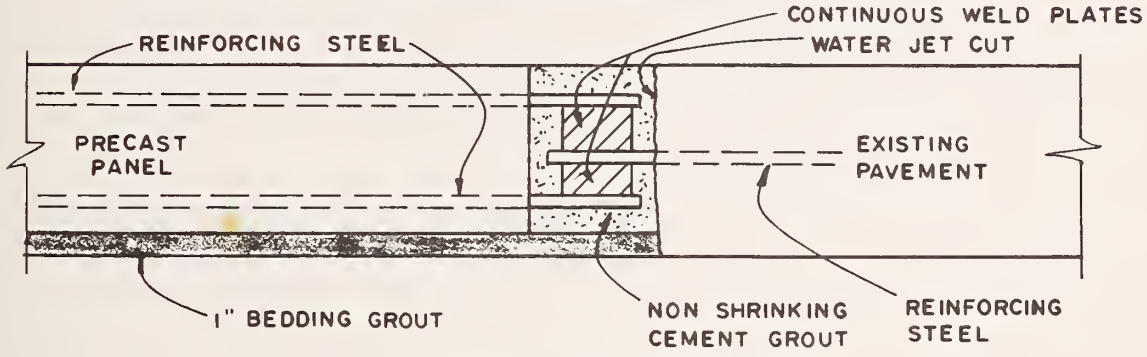
Joint Detail C (Figure 8-15) shows a proposed expansion joint in JRCP. Construction of the joint is similar to the rigid joint except that the tie bar provided from the existing reinforcing steel will be of adequate length to form a bond with the epoxy grout. This bar will be bent as shown in the Partial Plan View to provide the length of bar required within the joint. Smooth coated dowels will be cast into the precast panel such that they extend into the space provided as shown. Dual, upper and lower dowels are proposed rather than single dowels at mid-point of the section to eliminate aligment problems with the existing bars during installation and to provide an interlock system with the bent reinforcing bars extending from the existing pavement. A preformed joint filler material, as shown in section detail C, will be incorporated into the precast panel during casting. This preformed filler will include a removable strip along the top edge such that, when removed following placement of the epoxy grout, a space will be provided for either a premolded or poured rubber-asphalt joint sealant.

8.5 Utility of Potential Rehabilitation Techniques

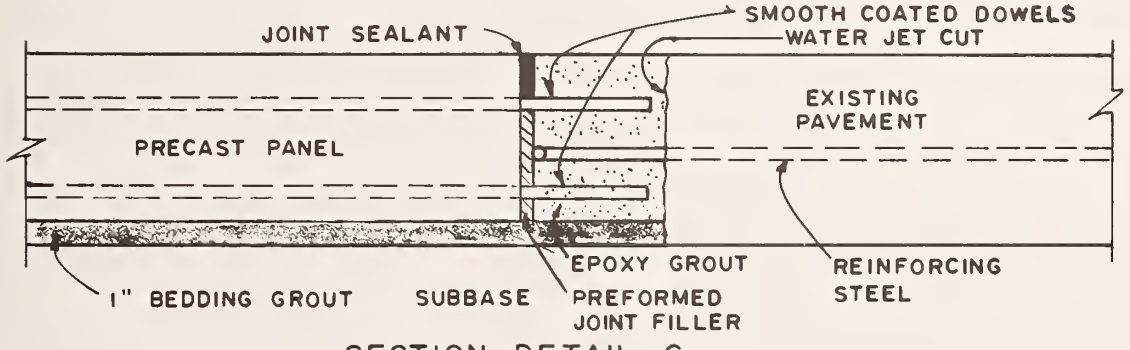
The applicability of existing and potential rehabilitation techniques to



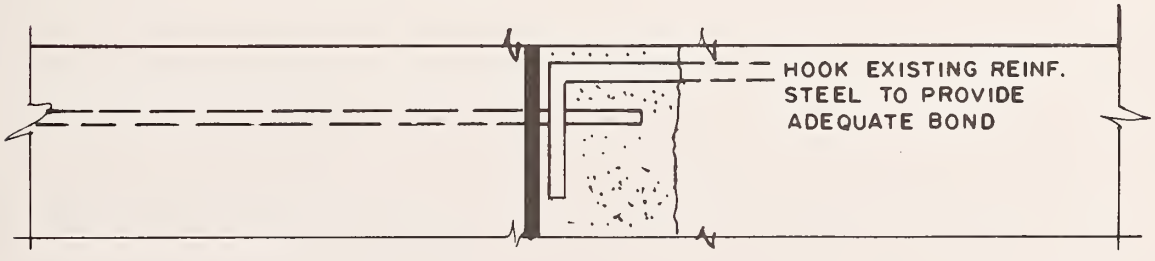
JOINT DETAIL A



JOINT DETAIL B



SECTION DETAIL C



PARTIAL PLAN VIEW

JOINT DETAIL C

Figure 8-15. Proposed Joint Detail for Precast Panel Pavement Rehabilitation.

1 in. = 2.54 cm

solve specific distress manifestations is shown in Table 8-7. To facilitate a comparison of these techniques the following assumptions were made:

1. Application of the techniques was limited to (a) urban areas with a traffic volume of 9000 veh/day at the time of construction increasing to 13.000 veh/day in ten years and (b) rural areas with initial traffic volume of 3000 veh/day increasing to 6000 veh/day in ten years.

2. The Serviceability Index (SI) of the pavement increased from 3.0 to 4.0 with application of the technique.

3. The existing pavements were reinforced or non-reinforced jointed concrete pavement.

4. Construction time and costs were based on averages for projects reported by several states.

5. Partial depth repairs were based on application to 1% of a lane mile of roadway (1.6 km).

6. Full depth repairs were based on application to 4% of a lane mile of roadway (1.6 km).

The utility values for each technique reported in Table 8-7 are valid only under these assumptions for specific conditions.

The partial depth precast slab technique of repairing spalled areas at transverse joints has a utility value approximately equal to that for the cast-in-place technique for both the urban and rural areas.

The costs of construction and maintenance for the partial depth precast techniques are lower than for the cast-in-place, however user savings are also lower due to better performance of the cast-in-place repairs. The precast technique would be preferable where initial cost is the primary consideration and the cast-in-place technique where long range performance is the governing factor.

Evaluation of the full-depth technique resulted in higher utility values for the precast methods in both urban and rural areas. The utility value for the "plain joint" design is slightly higher than that for the "tied joint". This is due to greater difficulty in reworking the "tied joint" technique even though its expected life is longer and the user savings is greater than for the "plain joint" method.

Table 8-7. Utility Values of Pavement Rehabilitation Techniques Using Precast Concrete Elements

REHABILITATION TECHNIQUE / DISTRESS MANIFESTATIONS	URBAN			RURAL		
	Spalling at Transverse Joints	Blow-Up/Compression Failure	Slab Faulting at Transverse Joints	Spalling at Transverse Joints	Blow-Up/ Compression Failure	Slab Faulting at Transverse Joints
A. Potential Techniques						
1. Full-Depth Precast Elements (Plain Joints)		0.789	0.789		0.763	0.763
2. Full-Depth Precast Elements (Tied Joints)		0.773	0.773		0.752	0.752
3. Partial-Depth Precast Slabs	0.660			0.728		
B. Current Techniques						
1. Full-Depth Cast-in-Place		0.676	0.676		0.655	0.655
2. Partial-Depth Cast-in-Place	0.663			0.702		

The results of this evaluation show a definite potential for the full-depth precast techniques of pavement rehabilitation. Further study and development of these techniques would provide an effective means of repairing blow up/compression and faulting distresses in rigid pavements.

9. Prestressing Existing Concrete Pavements

9.1 Introduction

Joints that perform improperly, or not at all, and transverse cracking rank near the top in problems with rigid pavements. These problems are discussed in Appendix A of this report.

Prestressed concrete has the ability to overcome many of the joint and crack problems that exist in rigid pavements. It is a material that is designed to virtually eliminate tensile stresses and cracks that one associates with them. The construction of prestressed concrete highway pavements in the United States is in its infancy, such as it was in the building industry in the early 1950's. The technology has not advanced to the point that it is fully accepted by highway engineers. A number of experimental sections have been constructed in various parts of the United States but none are longer than a few miles. A record of all of these, as well as installations in other countries, appears in Appendix D. There is no record of any attempt to rehabilitate an old pavement by means of prestressing.

The problems encountered in developing an effective plan for prestressing existing concrete highway pavements are far greater than those met in a new pavement designed for prestressing. Providing for the protection of the prestressing tendons, proper placement of the tendons, and overcoming the drag at the interface of the slab and the subbase are three of the major problems. Costs of installation are higher than they would be if original planning had been made for prestressing. The problems and their proposed solutions are discussed in the details of proposed methods later in this Chapter.

9.2 Prestressing Existing Pavements

Assuming that there is no serious deterioration in the existing pavement, the three major characteristics of the pavement that must be considered are slab thickness, concrete strength, and interface drag between pavement slab and subbase.

The thickness, if adequate for plain or reinforced concrete, is great enough for action as a prestressed concrete slab. As a matter of fact, it is excessively thick from the point of view of prestressing. The interface

drag, a combination of friction and shear, increases with slab weight, and the drag on the thicker pavement during thermal changes is greater than it would be on a thinner section. The existing pavement will very likely be at least 8 in. (20 cm) thick whereas a corresponding prestressed pavement would probably be at most 6 in. (15 cm) thick. In this study the thickness is assumed to be adequate and no attention is given here to slab design.

The strength of pavement concrete is treated in Appendix D. It is shown that slab concrete gains strength with time, and tests of cores taken from pavements designed for 28 day compressive strength of some 3500 to 4500 psi (24.1×10^6 to 31.0×10^6 Pa) show 10 year strengths of 6000 psi (41.4×10^6 Pa) and higher. It is not expected that the strength of concrete in an existing pavement to be prestressed will be any problem.

Pavements expand and contract with variations in temperature, and temperature varies from the top of the slab to depths below the bottom of the slab. These temperature changes and gradients, and differences in thermal coefficients of expansion between concrete and subbase, cause differential movements. The difference in movement of primary concern here is parallel to the centerline of the roadway. The greater movement is in the concrete slab and it attempts to slide or drag along the subbase. The resistance to that movement causes tension or compression in the pavement slab, depending on whether the temperature is rising or falling. The drag forces are very highly dependent on the nature of the interface surface. Observations and tests (2, 3, 4) have been made to determine the drag or "friction coefficients" and those tests and values are discussed in Appendix D. The tests show that the force required to overcome the initial drag is much higher than that required after the initial movement. The design of prestressing is very dependent on the drag, and there are a number of different base treatments, all of which give different values. This study makes the assumption that the drag behaves as friction, and is dependent on the weight of the slab and a coefficient of friction.

There are other features that are of concern, but they are not considered to be as important as the three discussed above. The cost element, a very

important one, being largely dependent on the amount of prestressing steel used and the preparation for its installation, is reflected in the different methods. The ones that are considered to be excessive, on a comparative basis, are discarded.

9.2.1 General Methods The purpose for prestressing existing concrete pavements is to close existing transverse cracks and prevent others from occuring. Transverse prestressing is desirable to close longitudinal cracks but the cost is very high considering the benefits gained.

Prestressing tendons placed in slots or trenches cut in the pavement, can be used most efficiently in longitudinal prestressing when they are oriented parallel to the pavement length. In such an arrangement the full capacity of the steel is available for combating the forces that cause transverse cracking. The details associated with the installation of the tendons and their anchorages make it advantageous in some situations to arrange the tendons in a crisscross fashion such as shoe lacing. This arrangement transmits a component of the steel transverse to the pavement thereby giving it a transverse prestress, dependent on the amount of steel and its angle of orientation.

There are advantages and disadvantages to both the longitudinal and the diagonal orientations. The main disadvantage to longitudinal prestressing is the full-width cut that has to be made in the pavement for installation of anchorages and for the prestressing operation. Those breaks must be made at intervals for expansion and contraction joints, though, and it is only the spacing that becomes the problem with longitudinal orientation. The main disadvantage with the shoelace orientation is the increased amount of steel and the amount of concrete cutting needed for the installation of the steel. There is possibly a small advantage in the transverse component of prestressing offered by this system.

Tendons will work with the highest efficiency if their forces are coincident with the forces resisting their action. In an ideal pavement situation, if there can be any such situation in rehabilitation, the tendon would just balance the effects of base drag, bending forces, and direct prestress in the slab. This would place the tendon in the lower half depth level of the slab in most situations. Placement at mid-depth level would be a good compromise.

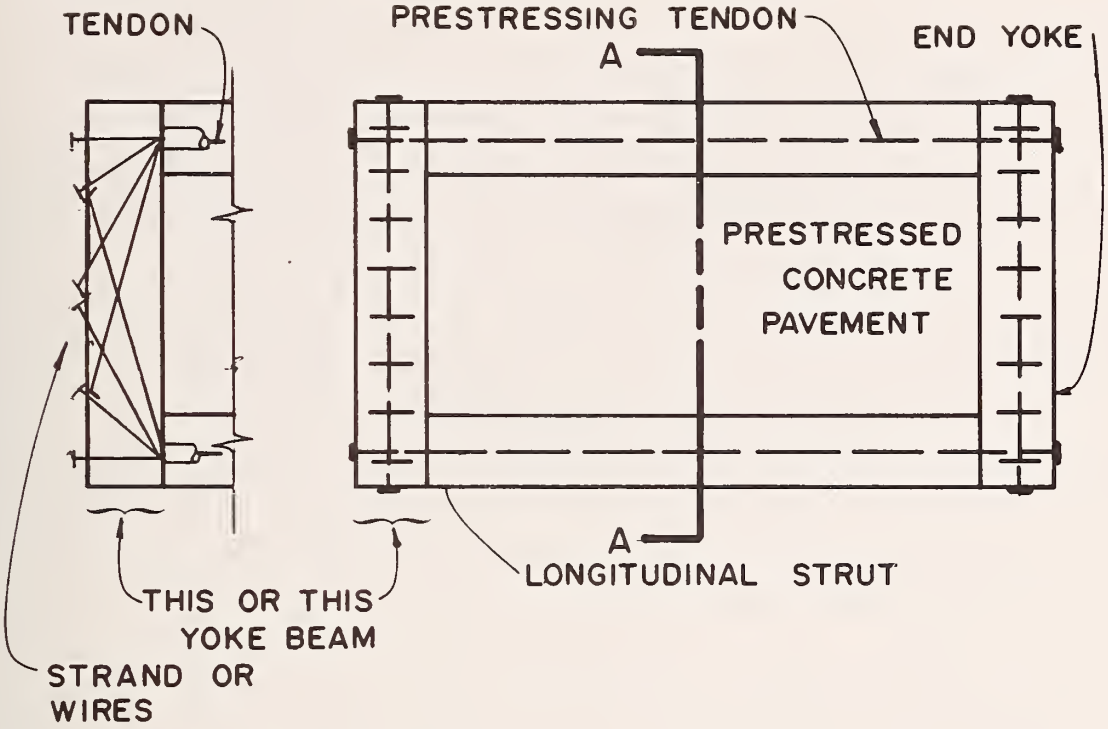
The tendon could be placed at approximately that level both inside the slab and exterior to the slab.

Installation of tendons inside the slab requires that trenches be cut in the existing concrete for the steel and then backfilled after placement, prior to prestressing. These tendons can be distributed across the width of the slab to whatever spacing that can be tolerated. The tendons, being encased in the element and distributed in the element, give favorable stress distribution and resist buckling tendencies as is normal in post tensioned concrete. This system is used in the plans proposed in this study.

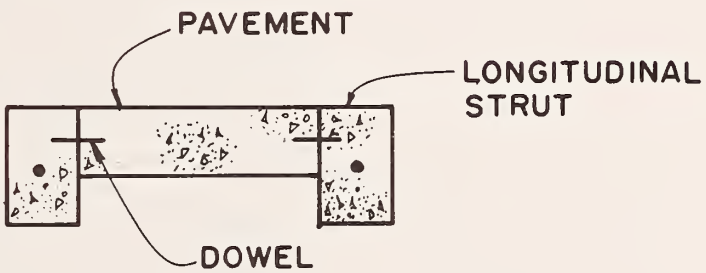
A system in which the tendons are placed exterior to the existing slab requires that they be placed to the side of the pavement in the present shoulders of the pavement system. These external tendons must be large ones because they must be concentrated in a relatively small space in members cast for the sole purpose of protecting them. The system, see Figure 9-1, consists of end yokes that carry the load from the tendons to the slab, the tendons themselves, and the members cast longitudinally along the side of the slab to protect the prestressing. The advantage of such an arrangement is that no cuts have to be made in the slab to accommodate the tendons. Disadvantages include: (1) The heavy yoke beams which must themselves be prestressed to enable them to carry the prestress forces, and reinforced to prevent shear failure, (2) The longitudinal struts must be doweled to the pavement as a preventative to slab buckling, and a large part of the tendon load is absorbed in prestressing them. These struts become exposed to traffic loads in service and their design becomes even more complicated in taking that into account. This system was studied and abandoned because of the costs and the reasons given above.

9.2.2 Details of Prestressing Systems Regardless of the system that is used, each must be able to overcome the drag of the pavement on its base when temperatures fall. If it is not overcome, the pavement will crack, or the cracks that are present will continue to open and close. Only enough prestress is necessary to overcome the drag; any more than that would add costs that are not warranted.

The survey given in Appendix D makes it very clear that there is much



PLAN



SECTION A-A

Figure 9-1. External Prestressing

uncertainty about the magnitude of the drag forces. The system studied here computes the drag as if it were friction. It uses a 9 in. (27 cm) thick pavement slab, a breakaway coefficient of "friction" of 3, and after the initial bond is broken reduces that coefficient to 1.5. The drag is assumed to be uniform, and no advantage is taken of reductions that might occur with time.

The strength of concrete given in Appendix D for pavements already in service appears to be entirely adequate, as mentioned earlier, for prestressing. The value of 4000 psi compressive strength was used here.

Time dependent strains in the concrete will be low. Shrinkage has already taken place and creep is low because of the low prestress as well as the age (5) of the concrete. Relaxation in the prestressing steel accounts for the major portion of prestress loss.

Design data and information are listed below:

Concrete at age 10 years:

Compressive strength, f'c = 4000 psi (27.6 MPa)
Shrinkage = zero
Creep = 321×10^{-6} per .45 f'c stress

$$\text{Creep} = [(321 \times 10^{-6})/(.45\ f'c)] \times 100 \text{ psi} = 1.78 \times 10^{-7}$$

where 100 psi (.689 MPa) is sustained prestress.

Coefficient of thermal expansion 0.0000050/°F
(0.000009/°C)

Prestressing Steel

Ultimate strength	270 ksi (1.86 GPa)
Modulus of elasticity	29×10^6 psi (200 GPa)
Prestress loss	AASHTO Code (6)
Tendon friction losses	(4, 10) K = .0002/ft (.00066/m)
Details and hardware	PCI Design Handbook (7)
	PCI Post Tensioning Manual (8)
Design stresses	ACI Building Code (9)
Temp. change, from installation	+50° (10° C)
	-70° (21° C)

9.2.3 Design Details Drag forces: Drag forces develop when the pavement attempts to contract. This is caused by lowering temperatures and resistance to movement at the pavement-subgrade interface. The forces vary along the length of the prestressed length as illustrated in Figure 9-2.

Assuming that the slab is continuously reinforced, T_{RS} will be about 40K (0.178 MN) at incipient cracking. The tendon force will then be

$$T_{ps} = (8.1 \chi - 40) \text{ k } [(3.60 \chi - 17.8) \times 10^4 \text{ N}]$$

Where χ is the distance from the end of the pavement section to the section of incipient cracking.

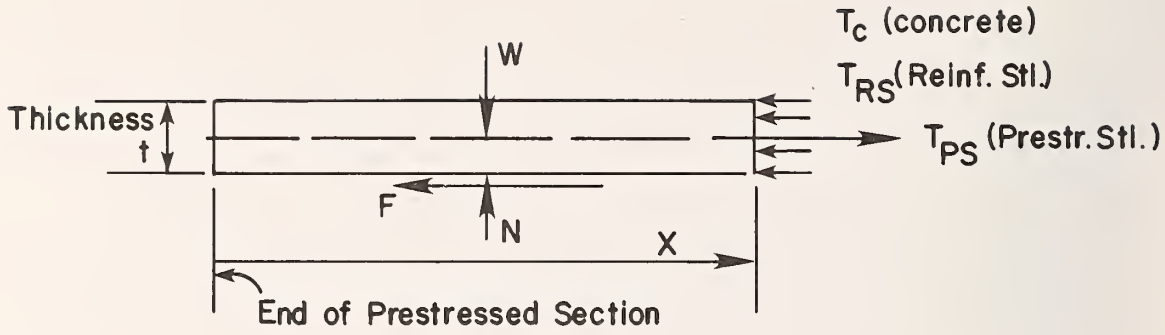
The tendon force, by this approach, increases linearly with the distance from the end to mid-length. A 200 ft (61 m) length would require 1620 - 40 = 1580 k (7.02 GN) tendon force at mid-length to break the initial bond between the slab and the ground. After it is initially broken, the drag would require 810 - 40 = 770 k (3.42 GN) tendon force.

The simple analysis above illustrates that it takes only a short distance to develop a very high force. This places severe restraints on the lengths of sections that can be prestressed. It does not appear feasible to introduce a bond breaker at the bottom of the slab, but it would be a great relief if it were possible. Bond can be broken with jacks (Figure 9-3) but great care must be taken against buckling. If the slab is already prestressed by tendons inside the slab, the danger of buckling is greatly reduced. A study was made of jacking non-prestressed pavement but was abandoned because of buckling problems.

Two methods that appear to be the most likely among several considered are outlined below under plans A and B.

Plan A: Prestressing with straight longitudinal strands set in trenches cut into the top of the pavement, Figure 9-4. The steps in the procedure are listed on the figure but they are explained here.

A section of pavement about 208 ft (63 m) long is chosen for prestressing, and the section is repeated as many times as is necessary to provide the required length of pavement. The length is limited by the drag between ground and slab, and the amount of prestressing steel that can be feasibly installed. Four tendons of four ½ in. (12.7 mm) seven wire, 270 ksi (1.86 GPa) strands,



Thermal Condition: Falling Temperature

$\Sigma F_H = 0 = T_C + T_{RS} + T_{PS} - F$

T_C = Tensile force in concrete = zero (cracked)

T_{RS} = Tensile force in reinforced steel

$= A_{RS} \times f_{rs}$

T_{PS} = Tensile force in prestressed tendons

$= A_{ps} \times f_{ps}$

W = Weight of concrete pavement = 2700 #/ft (39.4 kN/m)

t = thickness of pavement slab = 9 in. (22 cm)

N = W

F = drag force = $N \times C_f$

C_f = "coefficient of friction"

Initial $C_f = 3$

Subsequent $C_f = 1.5$

Figure 9-2. Forces in Prestressed Section During Contraction of Pavement.

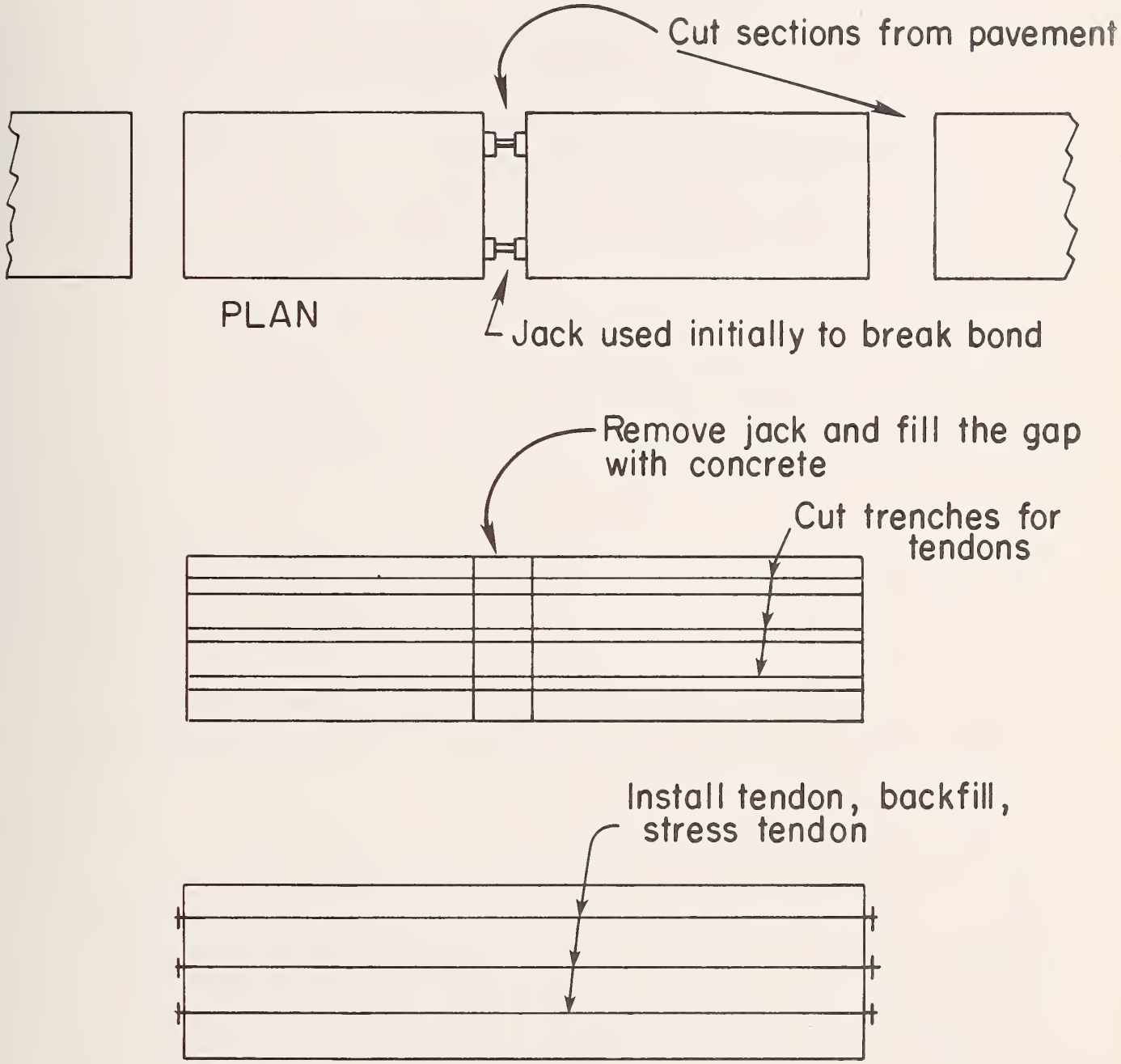
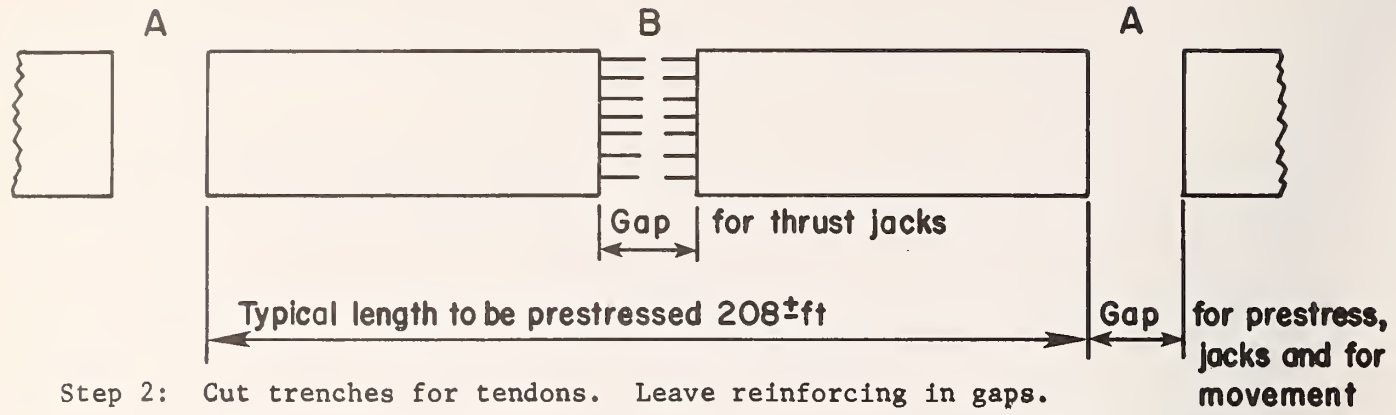

Cut sections from pavement
PLAN
Jack used initially to break bond
Remove jack and fill the gap with concrete
Cut trenches for tendons
Install tendon, backfill, stress tendon

Figure 9-3. Plan for Breaking Bond Between Slab and Ground, and Prestressing.

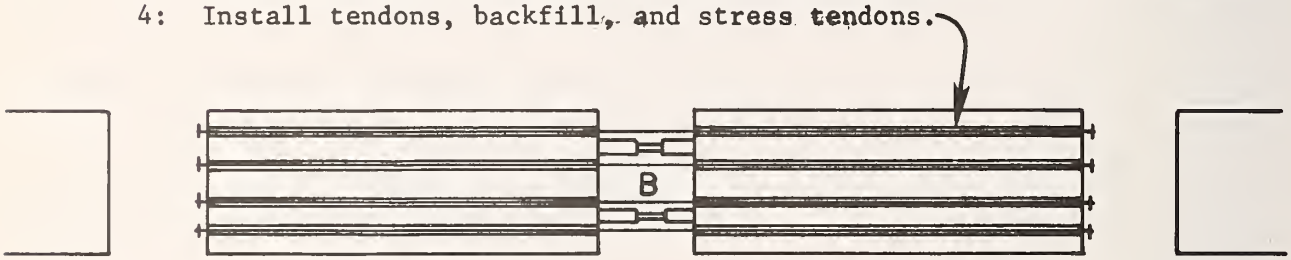
Step 1: Saw out sections to provide the gaps about 3 to 6 ft.



Step 2: Cut trenches for tendons. Leave reinforcing in gaps.

3: Install thrust jacks in gap at B to act as blocking until extended.

4: Install tendons, backfill, and stress tendons.



Step 5: Install temporary blocking at A allowing 1 in. gap for movement.

6: Extend jacks at B to close 1 in. gap at each end.

7: Block at B with adjustable blocks; remove jacks; splice bars at B; fill B with concrete.

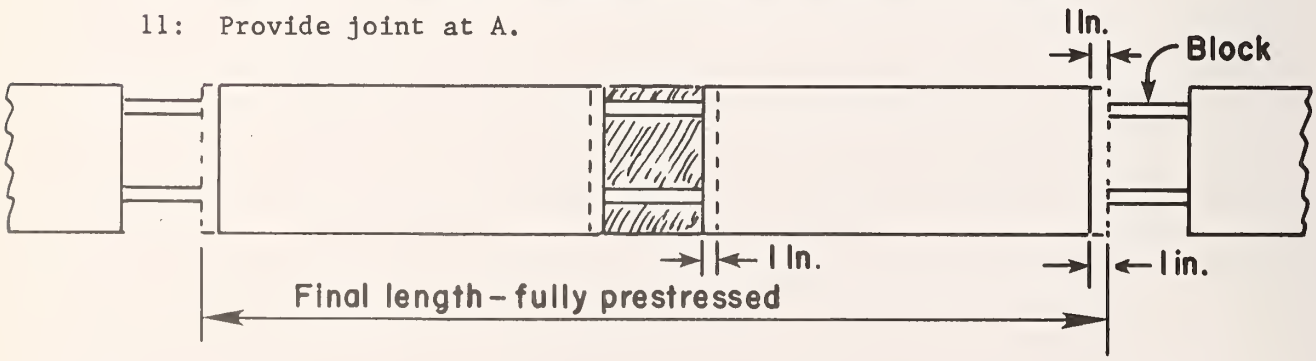
Barsplice at B.

8: Cure concrete at B; collapse blocks at B to prestress the concrete in gap at B.

9: Remove blocks at A.

10: Repeat for sections adjacent to the one just finished.

11: Provide joint at A.



1 in. = 2.54 cm

Figure 9-4. Prestress Plan A.

each with a total force capacity of about 400 k (1.78 x GN) after losses, are used. This is just enough force at mid-length of the section to overcome the drag between the mid-section and the end after the initial bond is broken with the subbase.

Initial bond is broken with heavy jacks installed in the mid-length gap. The jacks are extended only after the section has been prestressed to prevent any buckling that might try to develop. Blocking is provided at each end of the section, at gaps A in the figure, to insure that movement occurs in each half section and not in only one of them. Adjustable blocking is provided at mid-section gap B after bond has been broken. The jacks are then removed from B (expendable jacks could be used and left in place), and gap B is closed. Any longitudinal steel in the pavement will have been left in place, and it is now bent back into position, spliced, and encased in the gap filling concrete. The adjustable blocks at B are collapsed after the concrete has cured so that the concrete in gap B will be prestressed. Final touchup is given to B by filling spaces left when blocking is collapsed.

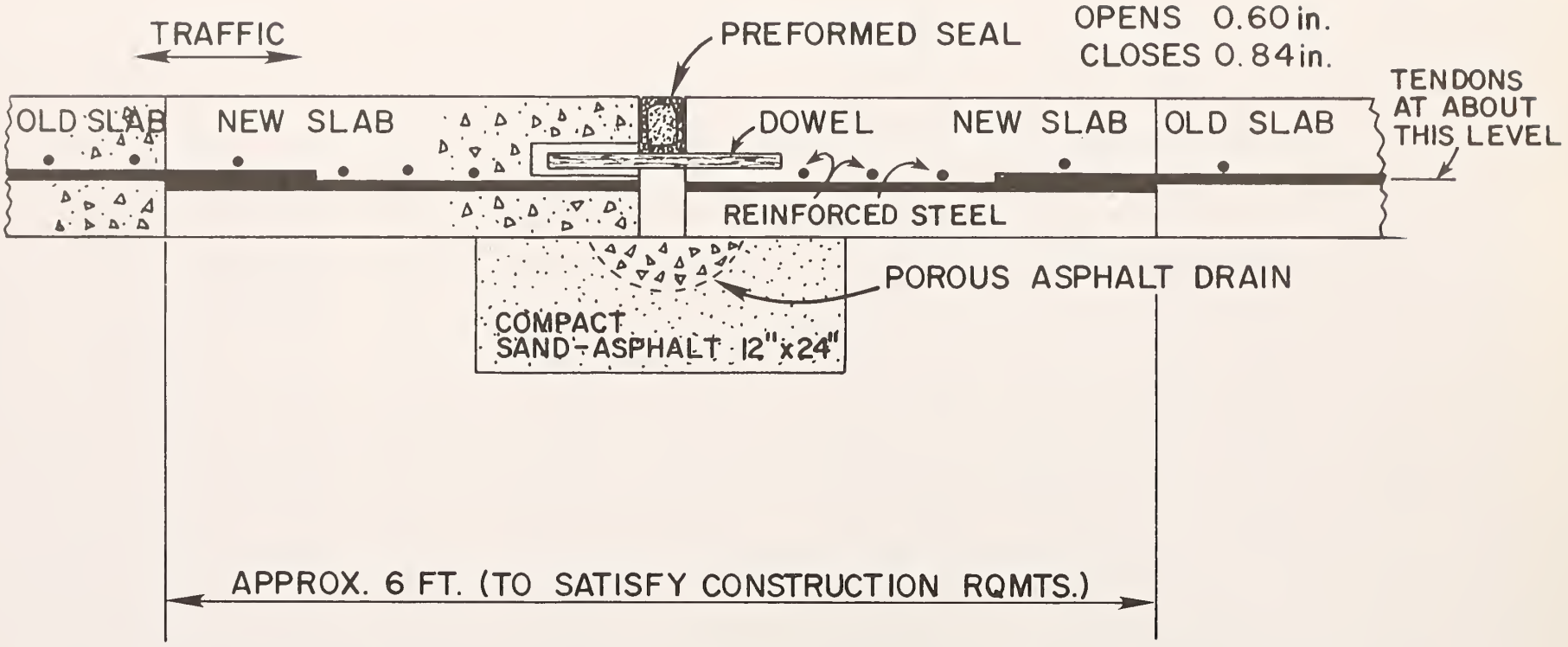
Joints for expansion and contraction, and for load transfer, are provided at gaps A after the adjacent sections have been prestressed. Features of that joint are shown in Figure 9-5.

The primary advantage of this plan, Plan A, is that a minimum of prestressing steel is used. The high bond is initially broken by the thrust jacks, and the lesser bond, one-half the initial value, can be overcome with fewer tendons than would be necessary if initial bond had to be broken by prestressing steel. Even with this initial bond break, the section cannot be much greater than 200 ft (61 m), unless some way is found for installing more tendons.

Disadvantages are believed to be the relatively short spacing of joints and the handling of the jacking and blocking at the mid-section gap. The joint design and the trenches for carrying the tendons are the same as in Plan B which is described below.

Plan B: Details for this plan are shown in Figure 9-6 and they are discussed here.

A section of pavement 340 ft (104 m) long is laid out for prestressing.



JOINTS ARE SPACED TO FIT REQUIREMENTS OF THE SYSTEM. SEAL HERE IS FOR SPACING OF APPROXIMATELY 210 FT.

(1 in. = 2.54 cm)

Figure 9-5. Joint Details.

This section is repeated as many times as is necessary to provide the required length of pavement. This plan uses jacks to break the initial bond, as does Plan A, in order to keep the quantity of prestressing steel to a minimum. Only enough prestressing steel is used as is needed to overcome the drag of the pavement on its sub-grade. The initial movement by the thrust jacks at each end of the section breaks the high initial bond and leaves it at an assured one-half of its initial value.

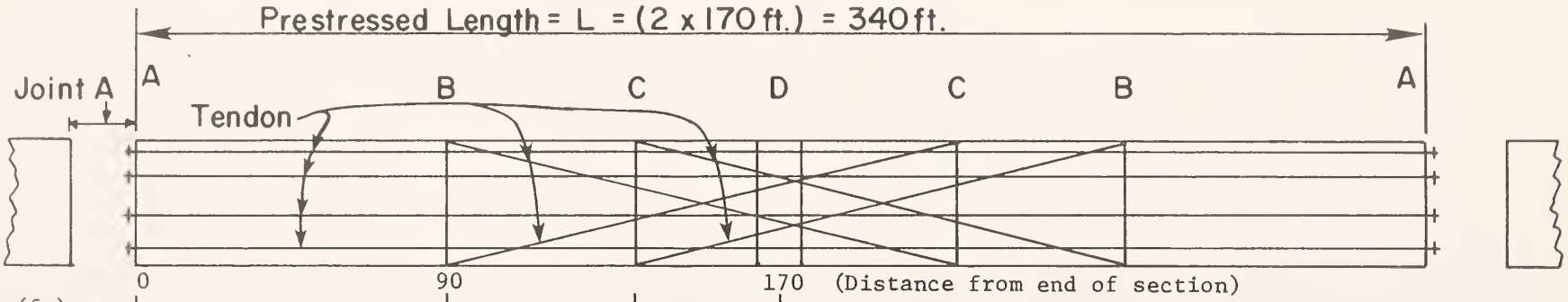
The feature of this plan which makes it different from some others considered is the variable amount of prestressing steel at different stations along the section. By referring back to Figure 9-2 it is seen that the prestressing force necessary to overcome base drag increases with the distance from the end of the section. The plan here fits the steel to the drag requirements; the greatest amount of steel at mid-length, and lesser amounts at stations nearer the end. This is the general principle used in leaf springs, plate girders, and the like. A combination of longitudinal and shoelace diagonal tendons permit such provisions.

The magnitudes of drag forces and tendon forces are shown in Figure 9-6. Drag forces, based on reduced bond, the high value of 8.1 k/ft (118 $\frac{KN}{m}$) having been broken by jacks, are less than prestress forces at all sections.

This system has the advantage of greater distance between joints than does Plan A, and the prestressing steel is arranged so that it is more efficient in its action of overcoming drag than is Plan A. Thus Plan B uses more prestressing steel than Plan A because of the greater length between joints.

9.3 Utility of Potential Rehabilitation Techniques

Five plans were studied for prestressing existing pavements but all except the two discussed above, Plans A and B, were discarded because of costs or details that were considered to be infeasible for good construction and performance. Plans A and B both use auxiliary jacks to break the initial bond between the pavement and the subgrade. Plan B has a joint spacing of about 170% of the spacing for Plan A. This is considered to be a distinct advantage in favor of Plan B because of the difficulties that have been encountered with joints in highways -- wherever they are. Estimated dollar costs per lane mile



Station (ft)							
Drag (KIPS)	0		364		547	688	(Drag after breaking initial bond)
Tendons	1	4		6	8		(Number of 4-strand tendons)
Tendon Force (KIPS)	447	390	613	570	788	748	(Total prestressing force after losses)

Steps:

1. Cut at A; remove 6 ft.± length for joints at A.
2. Cut out about 2 ft. of concrete at D. Leave longitudinal steel (12 in. on each side of cut).
3. Cut 4 longitudinal and 4 diagonal trenches about 4 in. x 4 in. for tendons.
4. Install tendons and tension them.
5. Place thrust jacks at A and move each section 1 in. toward D. Blocks at D should be installed to insure that proper movement is made, then after bond is broken remove them.
6. Splice reinforcing bars at D; fill at D with concrete and cure it. Then jack at A with thrust jacks to prestress the new concrete at D.
7. Remove jacks at A.
8. Install joint at A (joint in Figure 9-5).

1 in. = 2.54 cm. 1 k = 4.45 KN 1 k/ft = 14.59 k H/m

Figure 9-6. Prestress Plan B.

of Plan A is about 95 percent of that of Plan B.

Both plans call for trenches to be cut into the existing concrete -- some 4 in. (10 cm) wide by one-half the depth of the slab. These trenches are filled with concrete, cured, and prestressed with the pavement when the system is posttensioned. There is no history of performance of such an installation, and one can only estimate maintenance costs for the system. The fill concrete will be bonded to the trenches with a coating of grout -- or, if preferred, an epoxy bond coat -- and such bonding has performed well in concrete overlay and patching work. As a precaution in any first installations only salt-free pavements should be prestressed. When the service record of the installation has developed, one can make a better decision on its use on salted pavements.

The trenches cut for prestressing tendons run longitudinally in one of the proposed plans, and they run both longitudinally and diagonally in the other. When they are backfilled it is probable that they will be discernible because of imperfect blending, and they will likely cause slight surface irregularities. In order to eliminate these objectional details, a smoothing overlay of one-inch (2.5 cm) thick bitumuous material is specified.

The pavement should be better than it was when first placed because of the prestress and the high quality backfill concrete. Transverse prestressing was not called for because the literature did not indicate that longitudinal cracking is of any great importance. If transverse prestressing is to be used, there will be problems with details of crossing tendons, and costs will very likely increase very much.

Any patching that the pavement might require should be made to the pavement before prestressing. This section of the overall study assumes that the pavement is ready for prestressing and takes off from that.

The utility indexes for the two plans are given in Table 9-1 below along with those of a 6 in. (15 cm) thick AC overlay for purposes of comparison with a "standard" method of rehabilitation (see Appendix J for details). In evaluating the utility values of the prestressed concrete plans it was assumed that the proposed rehabilitation would bring the serviceability index up to the level of a new pavement. The requirement for frequent joints was

Table 9-1. Summary of Weighted Utility Values For Rehabilitation By Prestressing

Method	Utility
Prestress Existing Pavement: (Rural)	
208 ft sections	.545
346 ft sections	.549
6 in. (15 cm) HMAC Overlay	
(Suburban)	.696

considered to be the most serious deterent to long and trouble free performance of the systems. Joints have always been sources of trouble in jointed pavement, and prestressed systems have not been exceptions. Anticipated problems in this area weighed heavily in estimates of maintenance costs and serviceability indexes in evaluating the systems.

Another factor of high importance in the evaluation is development costs. The system has never been tried, and its acceptance will depend upon what is found from the installation and performance of trial sections. Since there is no way to make accellerated tests of a complete system such as these, the cost in time and money will be relatively expensive.

The overall utility of each of the two plans is considerably lower than that of the 6 in. (15 cm) thick bituminous overlays, which indicates the standard method of overlaying would be better -- for the assumed conditions. However this presupposes (1) a 6 in. (15 cm) overlay can be tolerated and (2) the prestressed pavement will only have a 15 year life. If an overlay cannot be installed (due to overhead clearances for example), or if the performance questions relating to prestressing can be solved to the point where the life can be extended, then the utility of this technique would be much higher.

The potential for eliminating the nagging concrete joint and cracking problems make this technique attractive. Even though, at this time, its utility is low, consideration should be given to rehabilitating an experimental section. Firm cost, performance energy and impact data could be obtained to more fully evaluate the overall utility of the technique.

10. CRACK REPAIR OF RIGID PAVEMENTS

10.1 Introduction

Cracks in concrete highway pavements are caused by volume changes in the concrete, stresses from traffic loads, or a combination of the two (1). Cracks reduce the strength of the concrete slab by reducing its ability to transfer load from one section to another (2). They reduce the strength of the system by allowing water to flow through to the subgrade material and reduce its capacity to support traffic loads. They contribute to a condition that supports corrosion of reinforcing steel (3). Since it is not possible to completely eliminate cracks, the pavement designer attempts to keep them narrow enough to keep them from leaking water, or he seals them. Among highway personnel, cracks are rated as the highest specific type of distress in in rigid pavements (4) (see Appendix A).

Cracks may be transverse to the center-line of the pavement, longitudinal, diagonal, random, or in a more or less grid pattern referred to as alligator or map cracking. These types, causes, and treatments are illustrated in Reference (5). Table 10-1 lists the distress manifestations and techniques for repair.

Transverse cracks are generally attributed to tension in the slab caused by temperature shrinkage. Longitudinal and map cracks are usually caused by stresses caused by wheel loads. In the more complex cracking, branches spread from a single crack which might be caused by volume change stresses or by heavy and repeated loadings.

The lowering of temperature in winter causes the concrete to shrink. The resistance of the sub-grade to shrinkage movement of the slab causes tension in the concrete. That tension in itself can be sufficient to crack the concrete, but traffic loads cause additional tension due to the slight flexing action. A slab that is not sufficiently thick, or a sub-grade that is not stiff enough, will cause cracking.

The resistance of the sub-base to slab movement is beneficial in that it prevents migration of the slab, but it is harmful in that it increases the frequency of transverse cracking. The resistance is commonly referred to as friction, but it is more likely shear resistance of the sub-base or a com-

Table 10-1. Crack Repair Techniques to Solve Specific Problems in Rigid Pavement

Distress Manifestation / Technique	Transverse Cracking	Longitudinal Cracking	Alligator Cracking
A. Potential Technique			
1. Pressure injection of epoxy resin	✓	✓	✓
2. Stabilize subgrade beneath the crack	✓	✓	
3. Pressure inject crack with epoxy and saw for new contraction joint	✓		
B. Current Techniques (5)			
1. Lowering water table		✓	✓
2. Remove excess water by subgrade drains		✓	✓
3. Increase shoulders slope		✓	✓
4. Lime injection		✓	✓
5. Repeat joint pattern	✓	✓	
6. Stress relief course	✓	✓	
7. Remt. upper layer	✓	✓	

bination of friction and shear resistance. The thicker slab reduces flexural stresses from traffic, but, being heavier, it develops greater friction in movement. Bonding between pavement and stabilized base, and mechanical interference to movement also add to resistance to movement.

10.2 Crack Spacing

Laboratory tests at the University of Texas (1) indicate that crack widths should be kept below 0.023 in. (0.58 mm) in continuously reinforced pavements to limit the amount of water that percolates through cracks. Spacings should not be less than 5 ft (1.5 m) nor more than 8 ft (2.4 m). The former limit helps to control corrosion caused by moisture, and the latter limits the loss of load transfer across the crack, among other things. Field surveys of transverse crack spacing in continuous reinforced concrete pavements in Illinois and Indiana (1) show that approximately 65% of the cracks are spaced less than 5 ft (1.5 m) apart and about 11% are spaced more than 8 ft (2.4 m) apart. Teng (6) reports average spacing of cracks found in a survey of 8-year-old continuously reinforced concrete pavement in Mississippi to be approximately 5 ft (1.5 m).

The spacing of transverse cracks in jointed pavement varies with the spacing of expansion and contraction joints, among other things. Interface drag, be it from shear or friction, or both, is always present when a pavement slab moves, and cracking takes place.

An 18 year old pavement in Minnesota is the subject of a report (7) on an experimental pavement constructed in 1940. In the plain concrete pavement sections, there were 10 times as many cracks at 18 year age than at 4 year age, and 30 ft (9.1 m) panels cracked about 8 times as frequently as did 15 ft (4.6 m) panels. The average spacing of transverse cracks was 14.4 ft (4.4 m) for 15 ft (4.6 m) panels, 18.7 ft (5.70 m) for 20 ft (6.1 m) panels, 20.2 ft (6.15 m) for 25 ft (7.6 m) panels, and 21.4 ft (6.52 m) for 30 ft (9.1 m) panels. The cracks include those preformed for contraction joints. For this installation, the best length for panels in reducing cracking, is 20 ft. (6.1 m). The report shows that there were approximately 310 ft/mi. (59 m/km) of longitudinal cracks. Crack widths are not reported, but a "typical transverse crack" is shown in a photograph and it is approximately 2/16 in. (3 mm) wide at the sur-

face.

De Young (8) reporting on 8 year old pavements constucted in Iowa in 1955 found that 20 ft (6.1 m) slabs had an average length of 19 ft (5.8 m) between cracks after 8 years of service. The average distance between cracks was 37 ft (11.3 m) for 50 ft (15.2 m) slabs, 37 ft (11.3 m) for 80 ft (24.4 m) slabs, and the slab with no joints had cracked to average length of 35 ft (10.7 m).

From all indications, the pattern of transverse cracking in continuously reinforced pavements develops eventually to a spacing of 5 to 6 ft (1.5 to 1.8 m). The spacing in jointed pavements differs with the length. The 20 ft (6.1 m) sections are reduced to about 19 ft (5.8 m), and those 50 ft (15.2 m) long and longer reduced to some 35 to 37 ft (10.7 to 11.3 m).

10.3 Crack Width

The limits on the width of a crack that can be tolerated depends on the ability of the system to transfer load from one side of the crack to the other, and on the amount of water that leaks through the crack.

If no steel crosses the crack, as in plain concrete slabs with no dowels, load is transferred from one slab to another by the base material, interlocking of the aggregates along the crack surface, or a combination of the two. As the crack becomes wider the effectivenss of the interlock between aggregate decreases. With this decrease comes greater dependence on the base to transfer the load. Repeated applications of load from heavy traffic gradually fatigues the base causing excessive flexural stresses in the slab near the crack. This can cause further cracking and general deterioration of the pavement in the local area. The continual rubbing of the two surfaces, such as under repeated loading, gradually wears down the interlock and reduces the effectiveness of the interlock (9). Both the distress of the base and the erosion of the material on the crack surfaces are evident in audible thumping and grinding sounds, as reported by field survey parties (10), when heavy wheels traverse the cracks.

Cracks provide a short path for water to reach the base of the slab. When the base becomes saturated it becomes weaker, depending on its character. It can reach the point where pumping develops, then deterioration becomes more

rapid.

Laboratory tests have been made to determine the effectiveness of aggregate interlock with cracks of different widths (2). Repeated load tests were made on plain concrete slabs on a 6 in. (15 cm) gravel sub-base. At 50,000 load cycles, one-half of the total number, a crack of 0.035 in. (0.9 mm) width was found to be 50% effective in load transfer for a 9 in. (23 cm) thick slab. At 0.023 in. (.58 mm) width the effectiveness is found, by interpolation from Figure 14 of the reference, to be about 80 percent. A slab on a stiffer sub-base, such as a stabilized material, would be more effective.

No upper limits of crack width have been found in the literature beyond which water percolation and subsequent softening of the base would follow. It would be preferable to have no cracks, of course, but the volume change nature of construction materials requires that cracks be tolerated. The upper limit of 0.023 in. (.58 mm) for steel corrosion considerations would probably satisfy requirements for a firm base and adequate load transfer from aggregate interlock.

The actual width of a crack in concrete pavement depends on a number of factors. In older pavements the cracks never close completely (11, 12). From the survey reported in Reference 1, Table 2.3, the average measured crack width in continuously reinforced pavements is 0.017 in. (.43 mm). Of the 930 cracks represented in the data, 28 sets of 6 cracks each, 168 cracks, had average widths of 0.026 in. (0.66 mm). This is greater than the 0.023 in. (.58 mm) upper limit. These wider cracks, (168/930 x 100 = 18% of the total), should be at least partially sealed.

De Young (8) shows no crack spacing as great as 1/16 in. (1.6 mm) for 20 ft (6.1 m) sections. For 50 ft (15.2 m) sections, he shows that all cracks are 1/4 in. (6 mm) or less in width, and for 80 ft (24.4 m) sections, none exceed 5/16 in. (7.9 mm). The average crack width for 50 ft (15 m) sections, computed from his data, is 0.109 in (2.77 mm), and for 80 ft (24 m) sections it is 0.088 in (2.2 mm).

The widths and spacing of cracks discussed above gives the basis for the values listed in Table 10-2.

There is no agreement among pavement designers that a satisfactory solution

Table 10-2. Crack Widths And Lengths Used For Evaluating Repair Methods

	Transverse		Longitudinal	
	Width (in.)	Number/Mile	Width (in.)	Length/Mile (ft)
CRCP	1/32	210	1/10	530
Jointed	1/10	35	1/10	310

1 in. = 2.54 cm; 1 mi = 1.6 km; 1 ft = 0.3 m

has been found to the problem of sealing pavement cracks, but there is agreement that cracks should be sealed. The effort to find the solution is a long and continuing one.

A crack that expands and contracts with temperature and moisture changes can be sealed only with a material that will work with the crack. It must accommodate the movement without losing its ability to seal. The material must also be able to prevent excessive intrusion of foreign solid matter, as well as water, into the crack. It is most desirable that no particles be permitted to enter. The material should be inexpensive and easy to apply. All of the joint sealing materials currently used possess one or more of the desirable characteristics. None possesses them all.

The popular materials for sealing cracks in concrete pavements are hot-poured and cold-poured bituminous materials, cold placed elastomers, and preformed seals (7, 13, 14). Some dissatisfaction has been expressed with the performance of hot-poured rubberized asphalt in the 1960's (13), but improvements in the material have made it a better performer (3). It has the advantage of being low in cost and easily applied, requiring no special equipment and little in the way of preparation. The preformed seal is recommended by McBride and Decker (15) for joint seals, but the meandering nature of pavement cracks that are not preformed would make its application difficult, if not impossible.

Pennsylvania tests (3) show that the preformed neoprene seal has a better performance record, at the date of the report, than either the cold placed elastomers or the hot-poured rubberized asphalt. The materials, as given in that reference, are listed below in order of the ratings and the material costs per 24 ft (7.32 m) lane are given for one joint size:

Neoprene	$24.19
Polysulfide	12.07
Improved rubberized asphalt	3.15
Urethane	5.47
Control (conventional rubberized ashhalt)	2.27

Additional information is given in Table 6-2 of Chapter 6 on the Repair or Replacement of Failed Joints.

10.4 Methods for Rehabilitating Transverse Cracks

Provisions must be made in pavements to accommodate volume changes due to shrinkage, moisture changes, and temperature changes. If the prepared joints

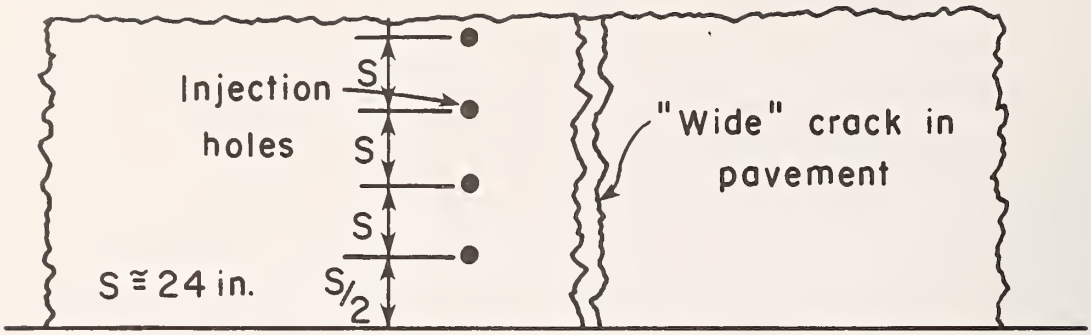
do not satisfy the requirements, cracks develop and become active. As long as their presence is required for these actions they must be tolerated, and they must be sealed to protect the system from the harmful effects of water.

10.4.1 Continuously Reinforced Concrete Pavements. Continuously reinforced pavements do not have joints for expansion or contraction, but they often do have cracks wide enough to permit water to percolate through. An estimate made earlier in this chapter indicated crack spacings in this type of pavement at 5 to 6 ft (1.5 to 1.8 m) with 18% of those being in excess of the tolerable width. This amounts to about one crack in five that is too wide -- one about every 25 ft (7.6 m) on the average. Since these cracks must remain active, and since they close almost completely at times, they must not be filled with incompressible materials. Two methods that would relieve this type of cracking are:

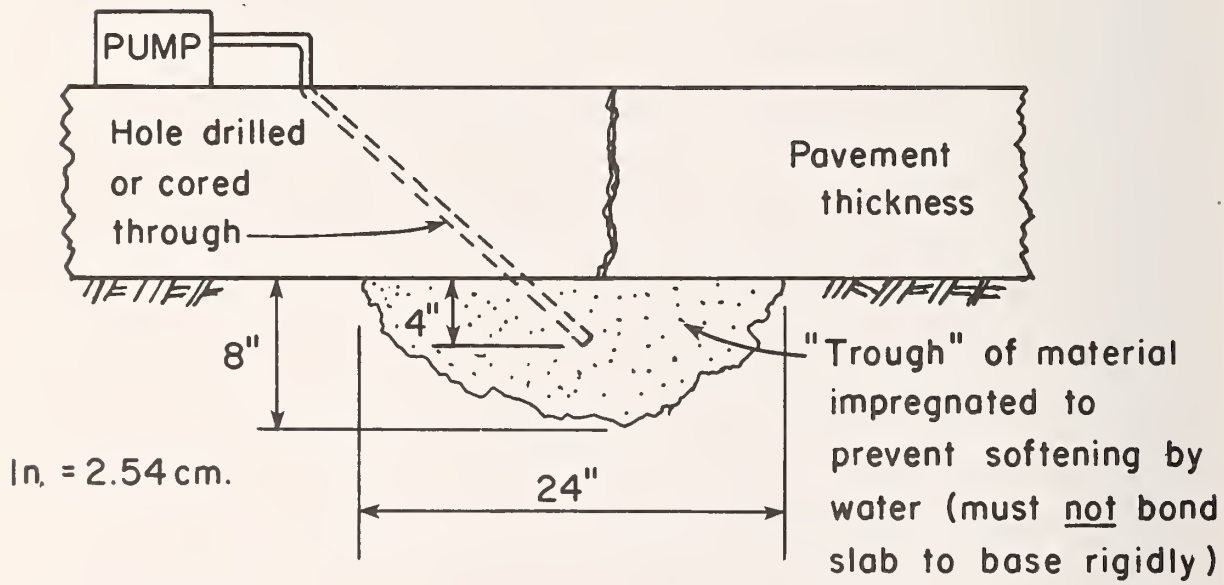
Plan 1. Penetration grouting of subbase beneath the wide cracks (Figure 10-1) so that a high modulus of base material is maintained under both wet and dry conditions. This would not protect the steel from corrosion (see Chapter 3 for complete discussion).

Plan 2. Pressure inject (16) the widest cracks, those wider than 0.023 in. (.58 mm) with epoxy resin. This should be done at the lowest temperature at which the resin will cure. It is estimated that the temperature will be approximately mid-way between the summer high and the winter low, and that the crack will be about half-way open. If heat is used in curing it should be held strictly to the local area of the crack, and it should be used only if absolutely necessary. No tension across the joint should be permitted until the resin is fully cured, which requires that no contraction in the concrete occur until full cure. When cured, the bonded section will be as strong or stronger than the concrete section if the joint surfaces are fully bonded.

In this plan, the contraction at temperatures lower than the injection temperature that would ordinarily have been absorbed by the bonded crack will be transferred to other cracks. Since the drying shrinkage will have already dissipated throughout the pavement, only temperature and moisture changes have to be handled. Assuming the bonded crack to be 1/2 x 0.030 = 0.015 in. (.38 mm), the width of the remaining 4 cracks (there is one wide crack for every 5 cracks), may be estimated as follows:



PLAN



SIDE ELEVATION

Figure 10-1. Plan for Injection of Stabilizer into Area Beneath Wide Cracks in CRCP.

Original average width =	0.917 in.	(.43 mm)
1/4 x 1/2 x 0.030 =	0.004	(.10)
New average width =	0.021 in.	(.53 mm)

This new average width is less than the critical 0.023 in. (0.58 mm).

This plan would permit the elimination of one wide crack, every fifth crack, without opening up the other cracks excessively, assuming that average conditions prevail, and that the distribution of the wide crack closure is distributed equally among all others. If there are a number of the wider cracks grouped together, all of them should be injected with epoxy during the lowest tolerable temperature condition for resin curing. Such action would leave possibly a relatively long jointless, crackless section immediately after the bonding. That concrete, having undergone full cure and years of service life, will have volume change due only to changes in temperature and moisture. It will form a new crack pattern and, since the bonded connection will be stronger than the concrete itself, the cracks should be better distributed and of more uniform width.

10.4.2 Jointed Concrete Pavements. Transverse cracks in jointed pavements, having been formed by contraction of the pavement, are very likely to be active during expansion and contraction due to temperature and moisture changes. These cracks can open up too wide for aggregate interlock to be very effective, and water can readily percolate through them. Their formation in the first place indicates the need for closely spaced contraction joints, and, since none were there, the pavement provided its own. These cracks are not wide enough in themselves to accommodate a sealer that will bridge the opening under repeated cycles of opening and closing. Two solutions are suggested for this situation:

Plan 1. Channel out the crack on the surface to a square-shaped section (5), and install hot-poured or cold-poured sealant (Figure 10-2). If the crack is straight, a preformed sealer might be installed. The seal will keep the crack, which is now effectively a joint, dry and the base material can more effectively transfer the load.

Plan 2. Clean the crack and bond the pavement sections together with epoxy resin injected into it under pressure until it is full. Provide a new contraction joint by sawing about one-third of the way through the slab. This sawcut should be as near to the injected crack as possible. Then, cut a

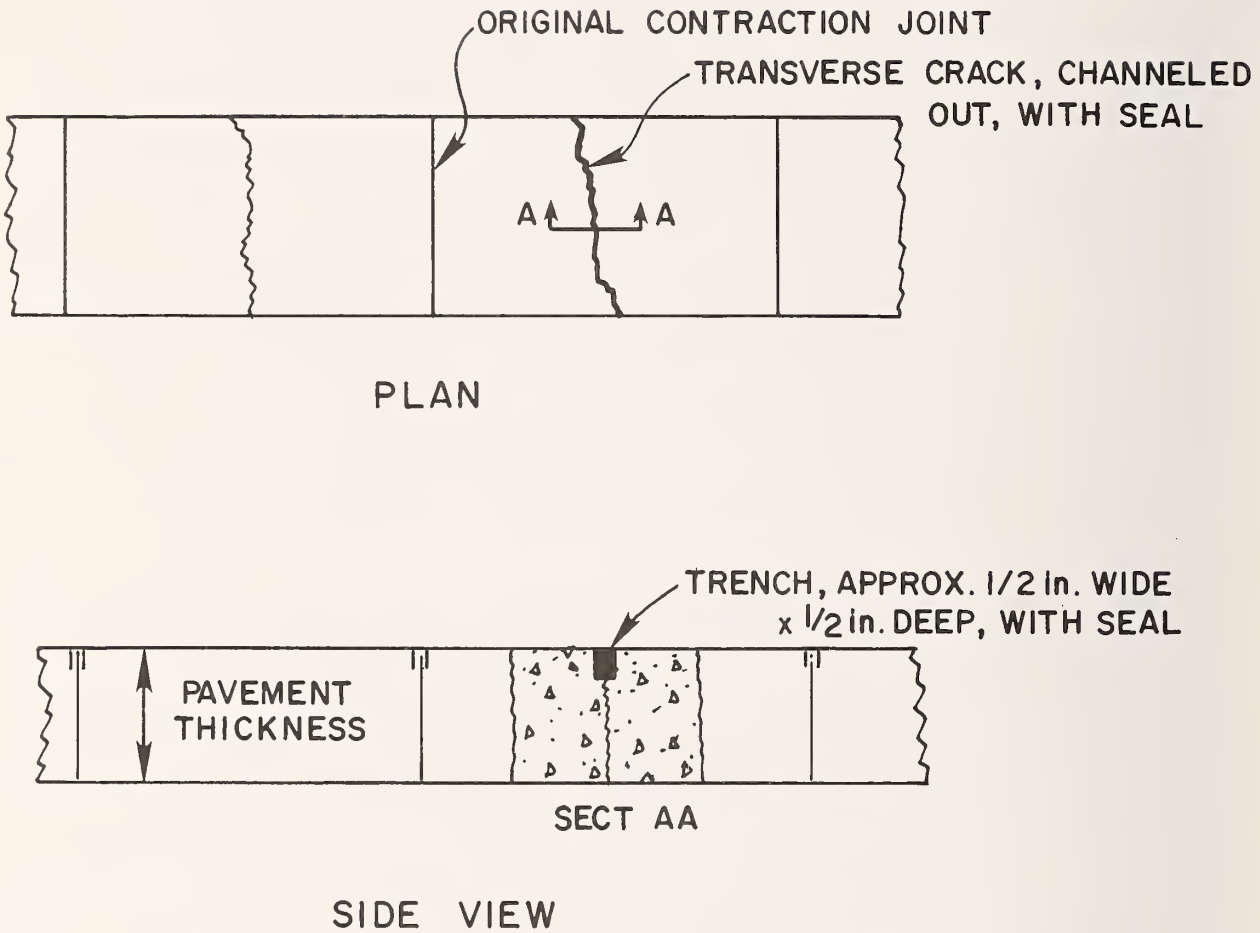


Figure 10-2. Repaired Crack in Jointed Pavement-Plan 1.

1 in. = 2.54 cm

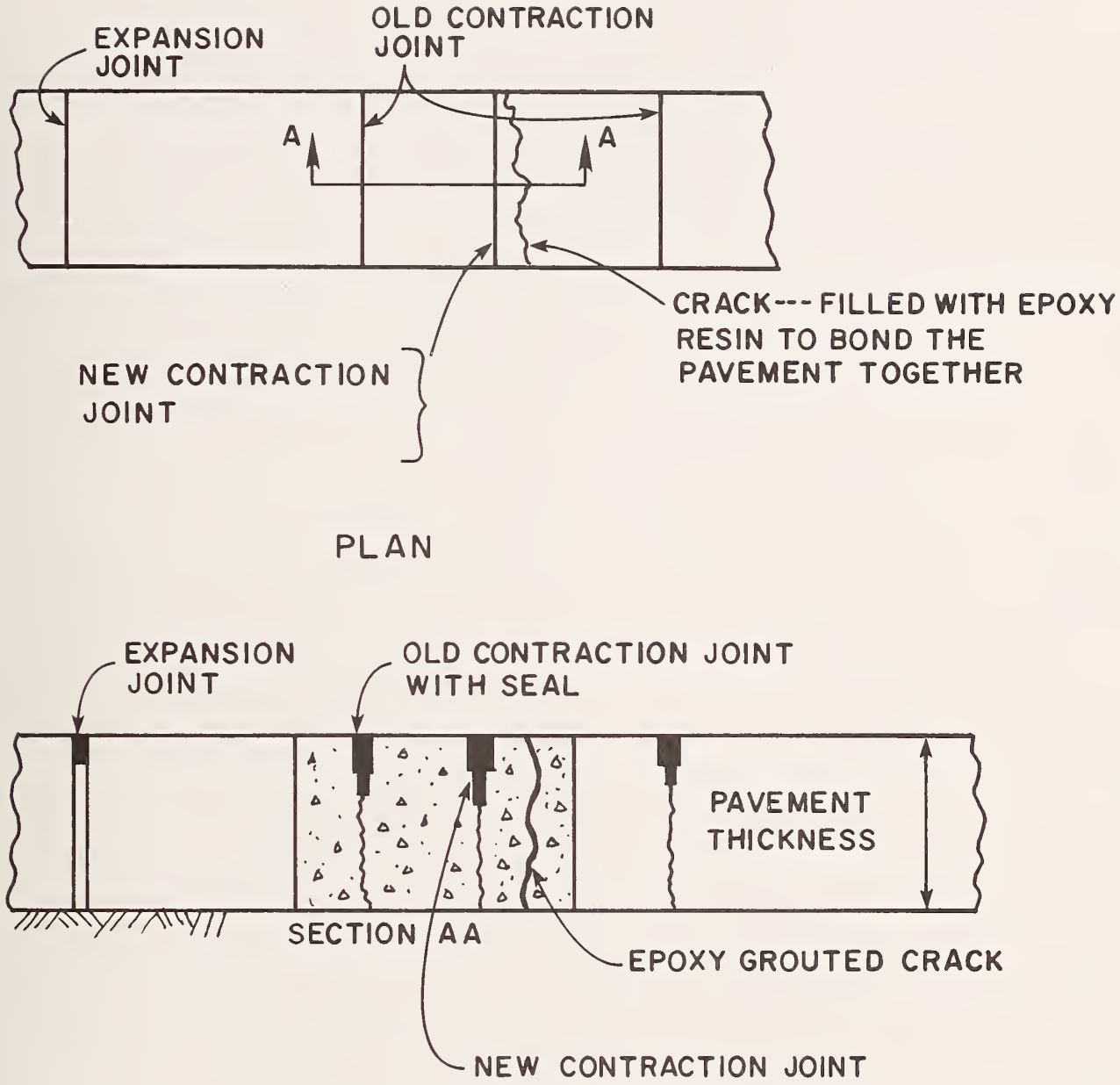


Figure 10-3. Repaired Crack in Jointed Pavement Plan 2.

square-shaped trench along the top bridging the sawcut, and install the seal along the rectangular cut (Figure 10-3).

No load transfer devices are proposed for these two plans. If the base is not stiff enough to transfer the load without excessive differential movement between the two sides of the joint, modifications will be necessary.

10.5 Utility of Potential Rehabilitation Techniques

One of the rehabilitation procedures discussed earlier, Plan 1 for jointed pavements, is not a new one. This procedure calls for routing out the top cracks and filling the trench with a poured joint-sealer. No record was found of installation of either of the other three methods.

In evaluating the procedures it was assumed that each would restore the pavement to a level of serviceability of a pavement approximately two and one-half years old. It was estimated that all except Plan 1 for jointed pavements would deteriorate to the same level the first five years after installation. The exception, Plan 1 for jointed pavements, was evaluated at a little lower five year serviceability because of difficulties that have been reported with the poured joint fillers.

Since transverse cracking is the primary source of crack trouble in both the jointed and continuously reinforced pavements, it--and longitudinal--cracking were considered to be potential problems in the future. Pumping was considered to be a potential trouble accompanying cracking.

Utility values for the four procedures are listed in Table 10-3. In the same table are listed values for a 6 in. (15 cm) thick AC overlay, and a 2 in. (5 cm) AC overlay, which are considered here to be current methods of rehabilitation. Results of the evaluation, listed in the table, favor any potential technique, except penetration grouting of the sub-base beneath cracks. Costs and energy consideration weigh heavily in favor of the plans proposed in this chapter.

For continuously reinforced concrete pavements (CRCP), pressure injection of epoxy resin in the wider cracks resulted in the highest utility value. For jointed pavements, routing and filling seems to be the best. Again it should be emphasized that the results are valid only for the assumed conditions.

Table 10-3. Utility Values for PCC Crack Repairing Techniques to Solve Specific Problems

Technique	Utility Value		
	Urban	Suburban	Rural
A. Potential Techniques			
Penetration Grouting of Subbase under wide cracks in CRCP	0.675		0.691
Pressure Injection of Epoxy into wide cracks in CRCP	0.754		0.772
Rout Out and Fill cracks in Jointed Pvt.	0.757		0.795
Pressure Injection of Epoxy followed by Sawing New Joint in Jointed Pvt.	0.756		0.755
B. Current Techniques overlay with 6 in. (15 cm) Asphaltic Concrete		0.696	
Overlay with 2 in. (5 cm) Asphaltic Concrete	0.753	0.737	

Generalizations beyond the specific cases evaluated may not be warranted. But, for the assumed conditions pressure injection of epoxy (for wide cracks in CRCP), and rout out and fill cracks (for jointed pavements) should be explored further.

11. RIGID PAVEMENT EDGE BEAM

11.1 Introduction

Rigid pavements often show signs of distress in the form of corner cracks or pumping resulting from the repeated application of heavy loads concentrated near the outside edge of the pavement. Heavy loads cause the pavement to flex creating excessive water movement through a hydraulic action and ultimately deteriorate base support capabilities. Pumping and curling of the pavement edge due to temperature gradients contribute to the loss of continuity between the pavement and supporting base resulting in high tensile stresses in the pavement when heavy loads are applied near the edge. Rigid pavement design theory suggests two methods of reducing these stresses; (1) widen the pavement so that the loads are concentrated farther from the edge and (2) thicken the slab in the area of the high stresses (Figure 11-1).

For many years pavement designers have used the technique of thickening the pavement section to reduce tensile stresses in the design of highway and airport pavements. The mid-section of runways as well as the ends for a distance ranging between 10% of the runway length to 1000 ft (305 m) are often thickened to reduce stresses in the pavement caused by a high concentration of aircraft traffic in those areas (1). Figure 11-1b and 1c show typical thickened sections of airport pavements.

Many of the early concrete highway pavements were 18 to 20 ft (5.5 to 6 m) wide and wheel loads were concentrated close to the outside edge. These pavements were typically constructed as 9-8-9 in. (23-20-23 cm), 9-7-9 in. (23-18-23 cm), and 9-6-9 in. (23-15-23 cm); being 9 in. (23 cm) thick at either edge and 8, 7, or 6 in. (20, 18 or 15 cm) thick through the mid-section. Figure 11-1a illustrates a 9-8-9 in. (23-20-23 cm) pavement.

The thickened edge technique of highway pavement construction was expensive due to the special subbase grading that was required. As pavements became wider, wheel loads were theoretically concentrated 3 to 4 ft (1 m) from the edge and the technique was discontinued except in special cases. But, under actual conditions large vehicle wheel

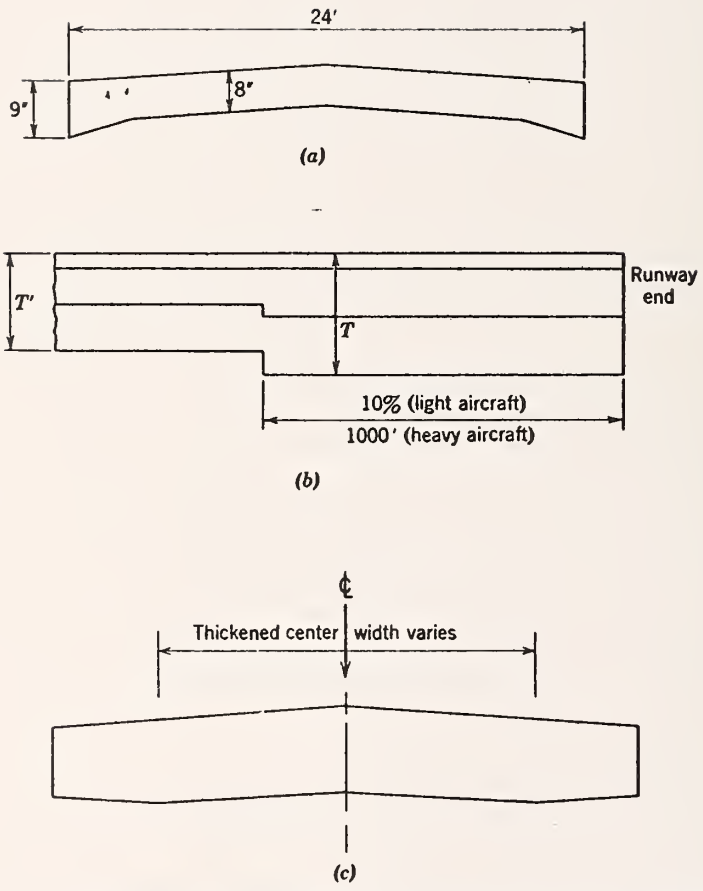


Figure 11-1. Thickened pavements for high load concentration. (a) Transverse section of 9-8-9 inch highway pavement; (b) longitudinal section of runway end (normal traffic, no channelization); (c) "keel" section for high load concentration on runway.

1 ft = 0.3 m

From Reference 1.

loads are often imposed on the pavement much closer to the edge than predicted and as a result can create distress manifested as corner cracking, longitudinal cracking or slab pumping (See Chapter 5).

11.2 Stresses in Rigid Pavement

Throughout its service life a rigid pavement is subjected to a wide range of stresses resulting from a variety of causes including wheel load, cyclic changes in temperature, changes in moisture and volumetric changes in the supporting material. A change in any of the factors tends to deform the slab often resulting in an increase in stresses, the magnitude of which can be related to the degree of loss of continuity between the slab and the base material.

Typical rigid pavement design strategy considers the concrete surface layer to be primarily responsible for carrying the loads applied and the primary function of the base materials to be one of maintaining the environment in which the pavement is constructed. The subbase serves one or all of the following functions (2):

1. To provide uniform and permanent pavement support.
2. To prevent or reduce frost damage.
3. To reduce the accumulation of water under the pavement by providing proper drainage.
4. To prevent pavement pumping.
5. To control volume changes in the subgrade and thus reduce the detrimental effects on the concrete slab.
6. To expedite construction.

Failure of the subbase to perform any of its designed functions will greatly reduce the service life of the pavement.

Assuming the subbase performs its tasks, the magnitude of the stresses created in a rigid pavement due to wheel loads are related to the deflection of the slab, the amount of which depends on the subgrade support and stiffness of the slab. The reaction of the subgrade per unit of area is expressed as a constant k in lbs/in^3 called the modulus of subgrade reaction. Although the value of k is proportional to the deflection, experience has shown that a wide range of k has little effect on the stresses in the concrete. A quantity relating stiffness of the

slab to the subgrade is called the radius of relative stiffness and is denoted as L in the following equation (3):

$$L = \sqrt[4]{\frac{Eh^3}{12(1 - \mu^2)\, k}} \qquad (11.1)$$

E is the modulus of elasticity of concrete, h is the thickness of the concrete slab and μ in Poisson's ratio.

The magnitude of the stresses created within a rigid pavement are not only related to wheel loads and changes in temperature and moisture but also to the point of application of the wheel loads on the pavement surface. The thickness of concrete pavement should be such that the maximum flexural stresses, whether created by wheel loads or differential temperature, will not exceed 50% of the modulus of rupture of concrete (2). The critical value of the modulus of rupture is generally accepted as that at 28 days under laboratory curing conditions.

In Westergaard's original rigid pavement design work three loading conditions were considered: (1) corner loading, (2) interior slab loading, and (3) edge loading. The corner loading condition exists when a load is applied over a small circular area with a radius of a, at a distance a_1 from the intersecting slab edges. For this condition the maximum tensile stress, σ_c, is calculated as:

$$\sigma_c = \frac{3P}{h^2}\left[1 - \left(\frac{a_1}{L}\right)^{0.6}\right] \qquad (11.2)$$

Where P = the wheel load in lbs.

a_1 = the distance (in.) from the slab corner to the point of load application.

h = slab thickness, in.

The interior loading condition exists when the load is applied at a point some distance from the pavement edge. The calculation of tensile stresses for this condition are somewhat more complicated due to the fact that the load is applied over a small area. Two slab theories are considered; the ordinary theory assumes that straight lines through the slab remain straight and perpendicular to the neutral axis under load. For the special theory this assumption is abandoned in the immediate vicinity of the load. The Westergaard equation for tensile stress due

to interior loads, assuming Poisson's ratio, μ, = 0.15, is:

$$\sigma_i = \frac{0.316\ P}{h^2} \quad [\ 4 \log_{10} (\frac{L}{b}) + 1.069\ ] \qquad (11.3)$$

Where b = the equivalent radius of the area of load application when the actual radius is less than 1.724 h.

The value of b is calculated using the following expression:

$$b = \sqrt{1.6a^2 + h^2} - 0.675\ h \qquad (11.4)$$

For large values of a, the ordinary theory applies and b = a.

In calculating the stresses for the edge loading condition, an equivalent radius "b" is used in place of the true radius "a", and is calculated by the same equation. The equation for tensile stress due to edge loading is:

$$\sigma_e = \frac{0.572\ P}{h^2} \quad [\ 4 \log_{10} (\frac{L}{b}) + 0.359] \qquad (11.5)$$

A comparison of the three equations show substantially higher stresses resulting from corner and edge loading as compared to interior loading and that the stresses decrease as radius, a, increases. Also note that a considerable decrease in stress can be achieved by thickening the slab.

11.3 Pavement Edge Support

The occurrence of pavement distress associated with edge failure can be substantially reduced by neutralizing the high tensile stresses induced by heavy wheel loads concentrated near the edge. Possible techniques capable of reducing these stresses include reinforcing the pavement edge, constructing concrete shoulders that are doweled into the pavement and constructing an edge support beam.

Handling of the tensile stresses by reinforcing steel placed in the pavement will not be considered in this project since this technique could best be applied at the time of initial construction.

Doweled concrete shoulders have considerable effect on reducing pavement deflection and increase pavement life. Researchers at the University of Texas in 1972 showed that doweled concrete shoulders reduced

pavement deflection by one-half and increased the service life of the pavement by 2 ½ times (4). Tests conducted by the Portland Cement Association revealed similar results. Figure 11-2 shows the load locations and resulting deflection measurements relative to the deflection measured at the center of the slab for a given load (It is assumed in this illustration that load transfer is provided across joints and cracks.)

Concrete shoulders are constructed either as an integral part of the paving or by being cast separately following the main roadway paving. Six in. (0.2 m) has been shown to be an adequate thickness (5). The shoulder is generally tied to the main paving using No. 4 (13 mm) or No. 5 (16 mm) bars 30 in. (0.8 m) long spaced at 30 in. (0.8 m) on centers.

11.4 Potential Edge Support Techniques

Doweled concrete shoulders reduce pavement edge deflection thus increasing its service life by, in effect, widening the pavement and causing the wheel loads to be concentrated farther from the edge. Another technique that may be feasible in achieving the same results is the cast-in-place pavement edge support beam. Figure 11-3 shows the proposed cross section of such a beam. Provided intimate contact between the beam and the pavement is achieved, the beam essentially increases both the thickness of the pavement near the edge and its width. The dimensions shown are arbitrary.

The effect of this technique on the tensile stresses in the pavement can be illustrated through the use of an example loading condition with and without the edge support beam. Assume a pavement thickness of 9 in. (23 cm) with a 9000 lb (4080 kg) load concentrated over an area with a radius of 8 in (20 cm) at the pavement edge as illustrated in Figure 11-4. Using the Westergaard equation for tensile stress under edge loading conditions the stress in the pavement can be determined from equation 11.5.

For a = 8 in. (20 cm), which is less than 1.724 h when h = 9 in. (23 cm):

From equation 11.4

$$b = \sqrt{(1.6)(8)^2 + (9)^2} - (0.675)(9)$$

$$= 7.46 \text{ in. } (19 \text{ cm})$$

Assuming $E = 3.0 \times 10^6$ lb/in.^{2} (2.1×10^2k Pa), $\mu = 0.15$ and $k = 50$ lb/in.^{3}, (1.4×10^6 kg/m^3), L = 43.94 in. (112 cm).

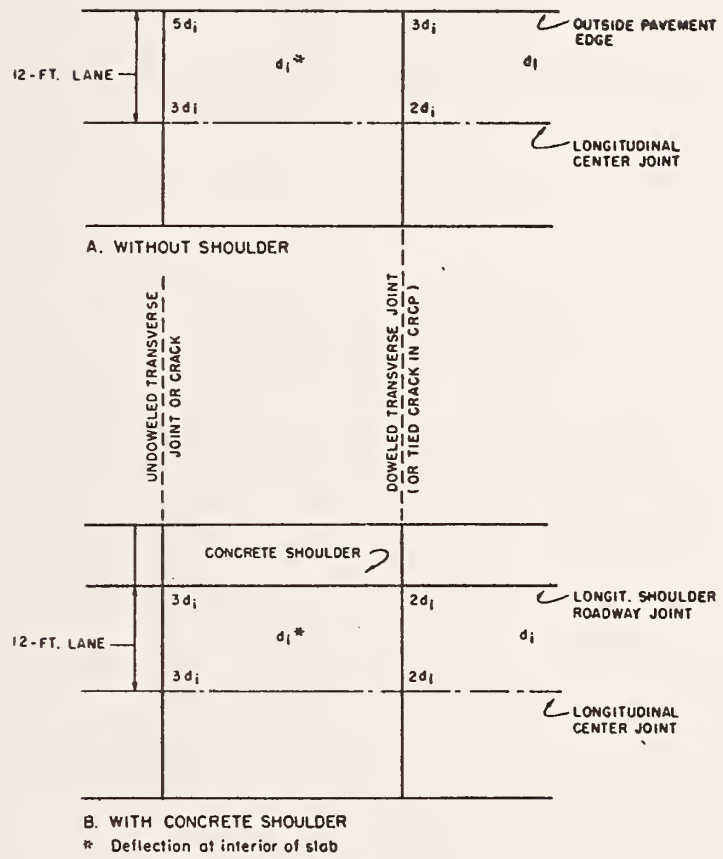


Figure 11-2. Deflections in concrete pavement as influenced by presence of tied concrete shoulder.

1 ft = 0.3048 m

From Reference 5.

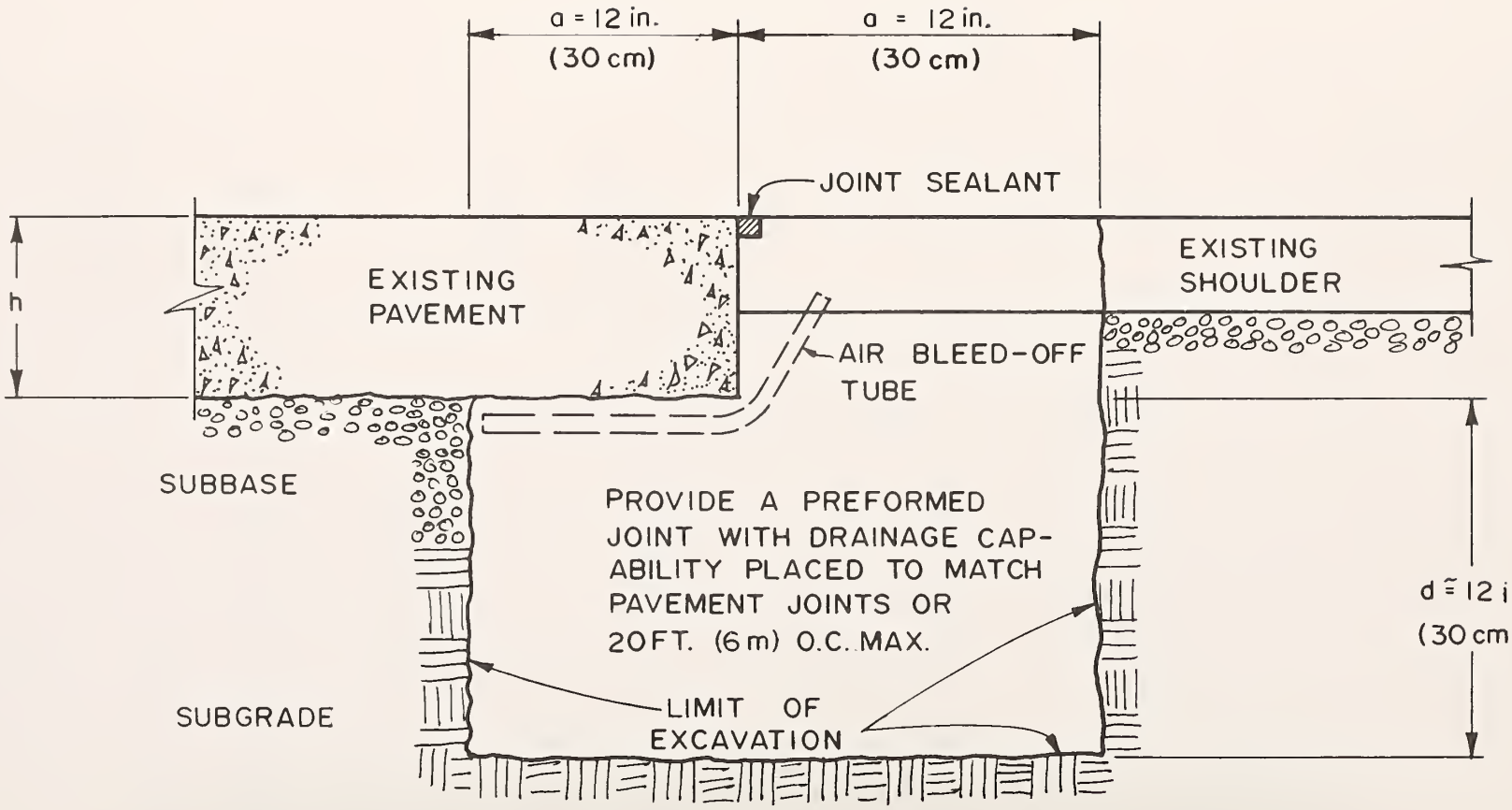
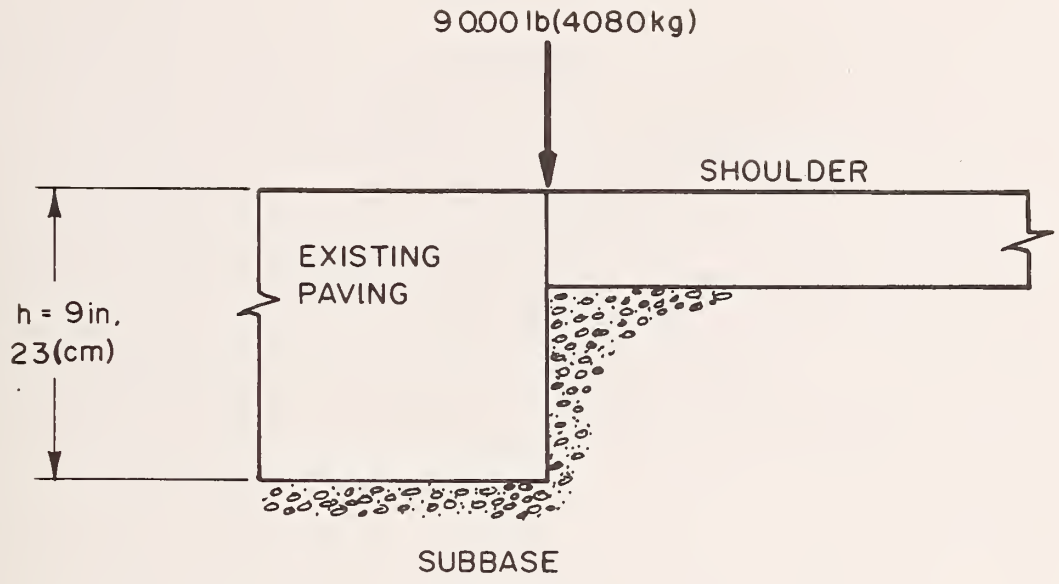
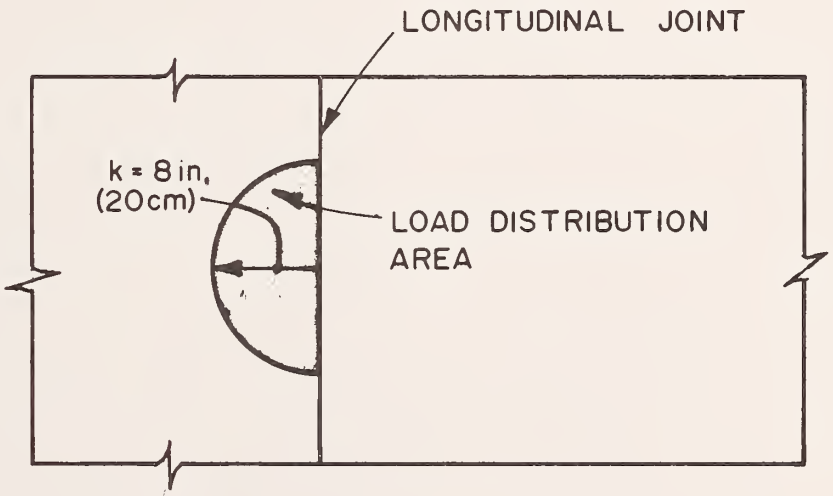


Figure 11-3. Pavement Edge Beam Detail.

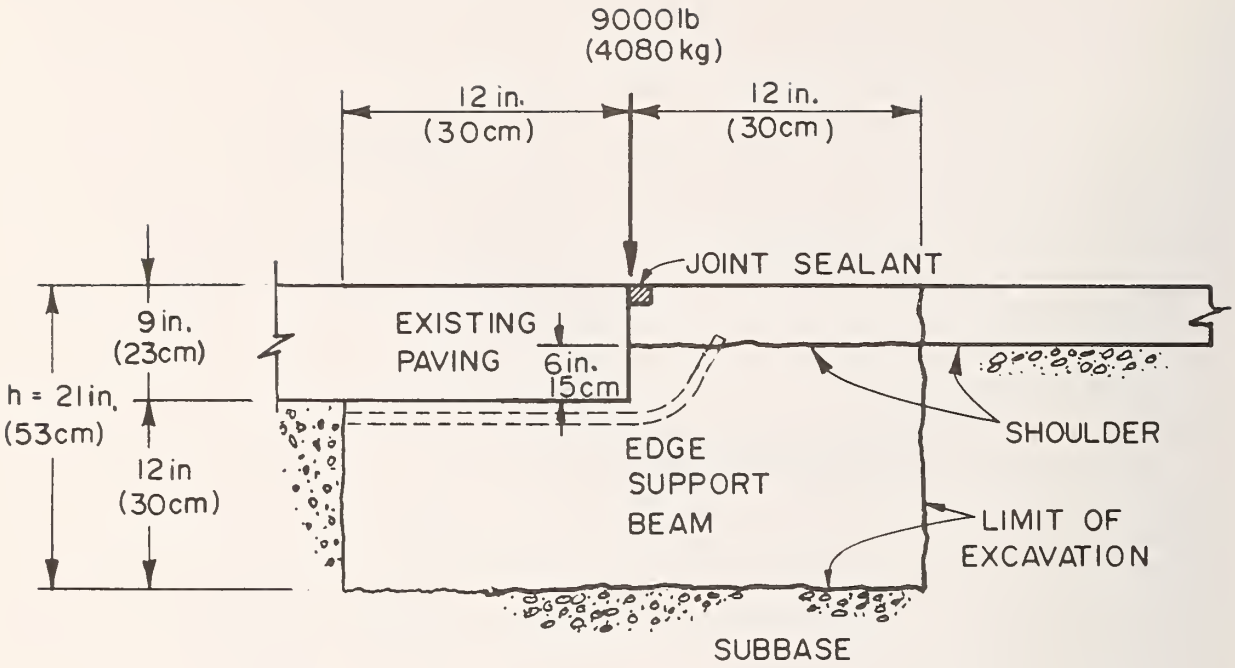


SECTION VIEW

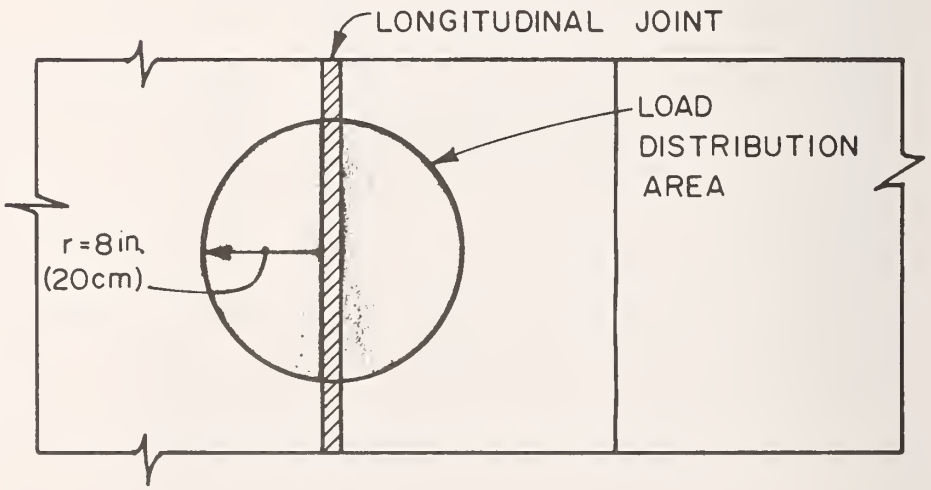


PLAN VIEW

Figure 11-4. Assumed Pavement Edge Loading Condition.



SECTION VIEW



PARTIAL PLAN VIEW

Figure 11-5. Assumed Pavement Edge Loading Condition (With Edge Beam).

From equation 11.5:

$$\sigma_e = 219 \text{ lb/in.}^2 \text{ (1500kPa)}.$$

The same load applied to a pavement with an edge support beam as shown in Figure 11-5 where h = 21 in. (53 cm) results in a stress as calculated below. The center of the load is no longer concentrated at the pavement edge so the equation for interior loading now applies (equation 11.3).

For a = 8 in. (20 cm), from equation 11.4:

$$b = 82.9 \text{ in. (210 cm)}$$

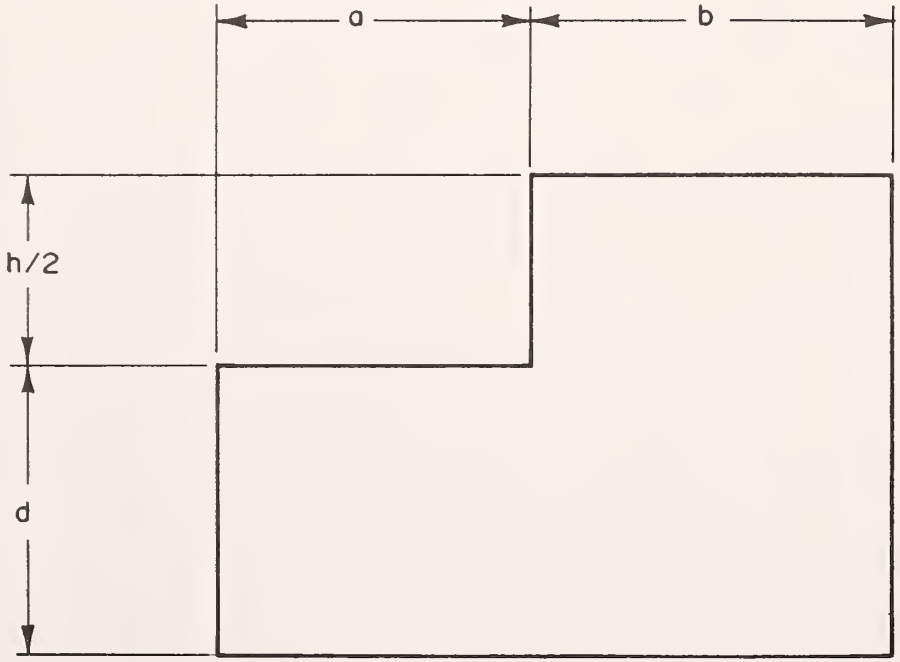
From equation 11.3:

$$\sigma_i = 32 \text{ lb/in.}^2 \text{ } (2.2 \times 10^5 \text{ Pa}).$$

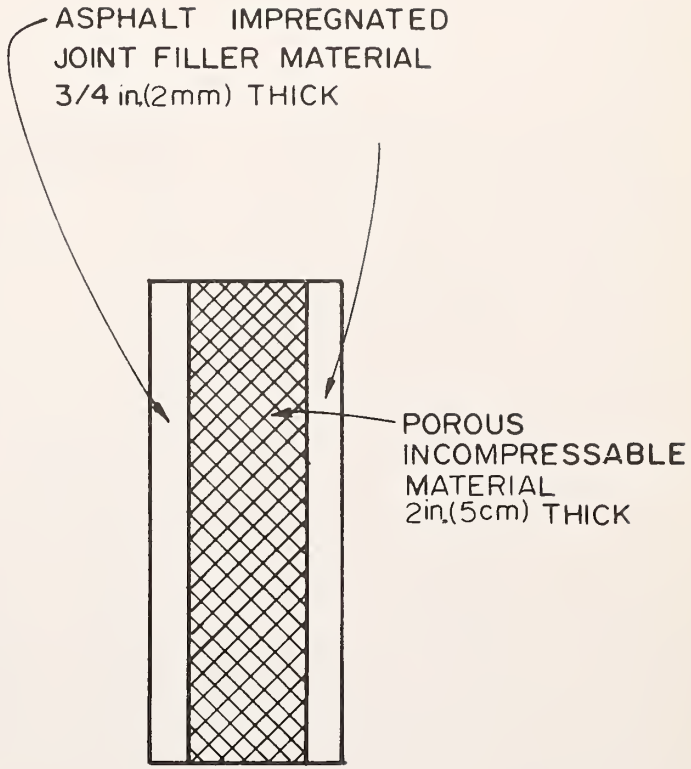
The addition of a 12 in. (30 cm) deep edge beam extending 12 in. (30 cm) under the edge of the slab and 12 in. (30 cm) outside the edge will theoretically reduce the maximum tensile stress in the pavement by 187 lb/ in.^2 $(1.3 \times 10^6$ Pa).

The actual construction of an edge beam of the type shown in Figure 11-5 would require the solution of several problems. Equipment for rapidly excavating the cross-sectional area as shown and cleaning the exposed bottom of the slab is not currently available. Provision for drainage of moisture from below the slab would be required. A technique for placing joints in the pavement would have to be developed. The most suitable material for construction of the beam would have to be selected and the best method for covering the exposed beam and sealing the longitudinal joint would have to be resolved. The optimum solutions to all of these problems are well within the capability of current technology and many are simply modifications of currently used techniques.

A potential solution to the joint and drainage problems is the placement of a preformed joint in the shape of the cross-section constructed of laminated material providing both differential temperature movement and drainage capabilities. The proposed joint is illustrated in Figure 11-6. The preformed joint would be placed in the excavation and firmly anchored prior to placement of the beams. Load transfer dowels would be placed through the joint as required.



ELEVATION



SECTION

Figure 11-6. Preformed Edge Beam Joint

11.5 Potential Materials for Edge Beam Construction

To afford an in-depth evaluation of the rigid pavement edge support beam several materials have been considered. The primary criterion used in the selection of the potential materials were:

1. The material must be easily prepared, handled and placed in the excavated area.

2. The strength of the in-place material must be comparable or higher than the concrete pavement.

The materials selected are discussed in the following paragraphs.

11.5.1 Portland Cement Concrete. The characteristics of PCC are well known. Its acceptance as a material for construction of pavement and grade beams is well established and thus will not be repeated here.

11.5.2 Sulfur - Aggregate Concrete. The combination of elemental sulfur, mortar sand and coarse aggregate has been found to produce a concrete with exceptional early strength properties. Compressive strengths in the range 5000 psi (34,500 kPa) in 6 hr is not uncommon (6).

Sulfur-aggregate concrete is prepared by mixing the components at ambient temperature in a typical portland cement concrete mixer and while mixing continues the batch is heated to approximately 285°F (141°C). The temperature to which the concrete is heated is dependent upon the intended applications. For an application such as the pavement edge support beam where the hot concrete is placed in contact with relatively cool material that presents a potentially rapid cooling rate, the temperature may have to be raised to as high as 320°F (160°C).

The physical properties and cost of sulfur-aggregate concrete is similar to PCC. Approximately 85 percent of its compressive strength is gained within the first few hours although its ultimate strength is in the same range as PCC (7). Table 11-1 shows the physical properties of sulfur-aggregate concrete. One potentially important advantage of sulfer-aggregate concrete is that after its useful life the materials can be broken up, remelted and reused without sacrificing any of its properties (8).

11.5.3 Pre-placed Aggregate/Grout Injection. Like PCC the properties of this material are well known. It is discussed relative to stabilization of materials in-place in Chapter 3 of this report.

Table 11-1. Physical properties of sulphur-sand-aggregate concrete mixtures.

AGGREGATE GRADATION Size No.	VOLUME RATIO Sulphur	Mortar Sand	Aggregate	AVERAGE STRENGTHS, psi Tension	Compression	Flexure
Crushed stone						
7	1	1.33	1.33	425	3150	595
	1	1	1.5	490	3550	570
	1	1	2	315	2800	505
57	1	1.33	1.33	425	4590	535
	1	1	1.5	460	4310	640
	1	1	2	375	4020	585
467	1	1.33	1.33	335	4150	525
	1	1	1.5	395	3530	525
	1	1	2	360	4320	505
357	1	1.33	1.33	280	2590	335
	1	1	1.5	370	3030	385
	1	1	2	375	3590	550
Gravel						
7	1	1.33	1.33	245	3150	490
	1	1	1.5	250	3000	450
	1	1	2	215	3660	510
57	1	1.33	1.33	310	4190	505
	1	1	1.5	305	3930	500
	1	1	2	325	3720	530
467	1	1.33	1.33	320	2960	410
	1	1	1.5	465	3050	420
	1	1	2	390	3150	405
357	1	1.33	1.33	295	3090	490
	1	1	1.5	300	3230	460
	1	1	2	305	3660	365
Crushed stone	Fine aggregate					
7	1	1.33	1.33	380	4020	505
57	1	1.33	1.33	425	5060	675
467	1	1.33	1.33	465	4620	625
357	1	1.33	1.33	495	4500	590
Gravel						
7	1	1.33	1.33	385	2940	465
57	1	1.33	1.33	360	4470	510
467	1	1.33	1.33	510	6190	585
357	1	1.33	1.33	495	5780	590

1 psi = 6984 Pa

From Reference (6).

11.6 Utility of Potential Rehabilitation Techniques

The pavement edge support beam technique has been evaluated assuming the use of several materials for casting the beam. Application of the technique to solve specific distress manifestations on rural and urban highways were considered based on the following assumptions:

1. Application of the techniques were limited to (a) urban areas with a traffic volume of 9000 veh/day at the time of construction increasing to 13,000 veh/day in ten years and (b) rural areas with an initial volume of 3000 veh/day increasing to 6000 veh/day in ten years.

2. The Serviceability Index (SI) of the pavement increased from 3.0 to 4.0 with the application of the technique.

3. The existing pavements were reinforced or non-reinforced jointed concrete pavements.

The utility values for each technique are given in Table 11-2. They are valid only under these assumptions for specific conditions.

Results of the evaluation indicate little potential for the edge beam techniques relative to tied concrete shoulders, even though preemptive techniques increased the life of the pavement considerably. The primary factor contributing to the lower utility value for the edge beam technique is the development and capital equipment costs assumed and the rehabilitation energy required. If further development of the edge beam technique resulted in a solution to the problems of excavation, its utility value would increase. However, it is doubtful that it would be greater than that for the tied concrete shoulder technique. Therefore there is little justification for further consideration of this technique at this time.

Table 11-2. Utility Values of Pavement Rehabilitation Techniques Using a Pavement Edge Support Beam

Distress Manifestations / Rehabilitation Techniques	Urban Areas	Rural Areas
	Slab Pumping and Corner Cracking	Slab Pumping and Corner Cracking
A. Potential Techniques		
1. PCC Beam	0.714	0.683
2. Sulfur/Aggregate Beam	0.647	0.591
3. Pre-placed Aggregate Grout Injection Beam	0.704	0.671
B. Current Technique		
1. Tied Concrete Shoulders	0.739	0.709

12. LOCALIZED REPAIR

12.1 Introduction

Localized repair is defined as spot rehabilitation of a distressed pavement (over a small area). This is constrasted to large scale rehabilitation, (termed segmental rehabilitation in this report) where the repair is executed over a large area (one lane mi, or 1.6 lane km, in this report). Most of the potential rehabilitation techniques evaluated in this project are applicable to both segmental and localized conditions, and thus they are presented and discussed in the applicable chapters.

There are a few potential techniques which are applicable to localized rehabilitation only. These techniques are presented and discussed in this chapter.

The repair of localized pavement failures is a problem that has faced the highway engineer for years. New techniques, equipment and materials for repair have been developed over the years, but, the problem of permanently repairing a locally failed pavement remains.

Conventional methods of repairing localized failures in asphalt concrete pavements include the following:

1. Fog Seal - Partial Width
2. Chip Seal - Partial Width
3. Surface Patch - Hand Method
4. Surface Patch - Machine Method
5. Digout and Repair - Hand Method
6. Digout and Repair - Machine Method

A general description of these types of repairs together with their suitability to economically repair certain types of flexible pavement distress are shown in Table 12-1. Costs for these repair activities can be found in Appendix C.

New materials and techniques have been developed for the digout and repair or replace method. A brief discussion of these new materials and methods is included below together with a new concept - super penetrating adhesive.

12.2 Digout and Replace

This method of repair has been utilized for centuries. The

Table 12-1. Conventional Methods of Localized Repair of Flexible Pavements

Maintenance Method		Type of Flexible Pavement Distress							
Descriptive Title	General Description	Rutting	Raveling	Flushing	Corrugations	Waves, Sags, Humps	Alligator Cracking	Longitudinal Cracking	Transverse Cracking
Fog Seal Partial Width	Light Application of Diluted Emulsion or a Proprietary Material Over a Partial Lane	P	G	P	P	P	F	P	P
Chip Seal Partial Width	Application of Asphalt and Cover Aggregate to a Limited Area	P	G	G	P	P	F	F	F
Surface Patch Hand Method	Application of a Premix Material to the Surface of the Pavement by Hand Method	F	F	F	F	F	F	F	F
Surface Patch Machine Method	Application of a Premix Material to the Surface of the Pavement with Machine	G	F	F	F	G	F	F	F
Digout & Repair Hand Method	Removal and Repair of Limited Areas by Use of Hand Tools	F	F	F	G	F	G	F	F
Digout & Repair Machine Method	Removal and Repair of Limited Areas by Use of Mechanized Equipment	F	F	F	G	F	G	F	F

Suitability of Method for Economically Repairing Distress

G - Good

F - Fair

P - Poor

techniques, equipment and materials have changed and have improved the likelihood of permanent repair. The most successful technique involves the removal of the deteriorated pavement, squaring-up of the area and replacing with a strong, durable material.

Equipment has been developed which greatly improved the removal and replacement procedures. Examples of this type of equipment can be found in reference 1, 2, 3 and 4. Additionally, a research contract will be let in the near future by the Federal Highway Administration for the purpose of developing a mechanized patching machine. The availability of new equipment has altered techniques used for patching.

Materials that are satisfactory for use in roadway patching operations must have certain economic and operational requirements as well as certain physical-chemical properties. A recent paper by Siemens (5) summarizes these requirements which are included below.

12.2.1 Economic Requirements

1. The cost constraints of the material are related directly to the effectiveness of the material as an all-weather permanent patch, and to productivity increase in man and equipment hours required per pothole repaired. A high percentage increase in material costs can be offset by a relatively low percentage decrease in labor costs.

2. The material should be more durable than standard emergency repair mixes. Durability is primarily related to its ability to remain in the pothole a minimum of two seasonal cycles. However, the exact duration required will depend on total cost/performance trade-offs.

12.2.2 Operational Requirements

1. Material application should minimize traffic disruption. Therefore, application procedures at the scene should be quick and simple, involving no more pieces of equipment or more complex application methods than current procedures. Barricades should be required only when a crew is at work at the scene. This implies that the material should bear traffic in about ten to twenty minutes after repairs are completed.

2. The material should be easy to store, and packaged in reasonably sized, easy to handle configurations.

3. The material should have a sufficient shelf-life; a minimum of one seasonal cycle.

4. The material should be useable in ambient temperature ranges from -20° to 120°F (-29° to 29°C).

5. The material should adhere to wet potholes with little or no pothole preparation required.

6. The material application should not be potentially dangerous to crew either in terms of toxic fumes, flash point, or other dangers to health and safety.

7. The material and its application should not cause significant air pollution or water pollution from leachate run-off.

8. The material should not require elaborate mixing and heating procedures at the scene. If possible, the process should use commercially available equipment.

12.2.3 Physical-Chemical Requirements

1. The material should be less susceptible to temperature variations for a number of properties such as flexibility, ductility, penetration and viscosity.

2. The material should have increased flexibility over a wide temperature range so that the patch is less affected by expansion and contraction or shifting of pavement surface and base structures.

3. The material should have adequate adhesion and bonding to edges of the pothole, or surface of pavement (in surface patch), and to aggregate and chips.

4. The material should have comparable resistance to both abrasion and penetration.

5. The material should have minimal permanent plastic deformation under stresses and strains, i.e., the stress/strain curve should be time dependent.

6. The material should be resistant to de-icing chemicals and at least as resistant to petrochemicals as surrounding pavement.

7. The material should not degrade under sustained heating for at least one work shift; similarly, material should not degrade significantly upon reheating the material at least once prior to application.

The search for this ideal material or materials has led to the development of numerous new products. Some of the materials developed have been highly successful while others are of questionable nature.

The more popular materials presently used in this patching operation include mixtures of aggregate with either asphalt cement, cutback asphalt or emulsified asphalt (6, 7, 8). Some of the new patching materials developed include those by Public Technology, Inc. (9), Pavon Corporation (10), Permagile Corporation of America (11), Republic Steel (12) and United States Gypsum (13). These materials are commonly mixtures of portland cement with such products as gypsum, magnesium ammonium phosphate or other chemicals; asphalt cements with such products as rubber, ethylene-vinyl acetate or other chemicals or epoxy type materials.

Permatop is a structural plastic allowing mortar-topping which is manufactured by Permagile Corporation (11). This material provides a hard-surfacing skid proofing and corrosion protection of either new or eroded concrete surfaces. The material develops good bond strength with the old concrete and provides protection from both physical and chemical attack. The primary application for this product will be for repair of spalled areas and scaling of portland cement concrete pavements and for bridge deck repair.

Republic steel has recently developed a patching material for concrete pavements (12). This material is produced from a reaction between a complex ammonium phosphate solution (APS) with magnesium oxide (MgO) to form a magnesium ammonium phosphate hydrate. Since magnesium is relatively expensive, a composition was developed by Republic Steel which used low cost limestone and/or dolomite aggregate together with fine magnesium to react with the liquid APS for bonding. The resulting mix sets in under 10 minutes. Crushing strength approaching 2,000 psi is obtained within 30 minutes and 3,500 psi in 24 hours. Equipment for mixing and placing the material in pavement application is not

generally available to highway maintenance crews.

A number of other products containing gypsum to promote rapid set have been utilized. An example of this type of material is Duracal which is produced by United States Gypsum (13). Detailed discussions of these types and other types of products can be found in an NCHRP report on Project 14-4 (14) and the NCHRP Synthesis on Pavement Rehabilitation Materials and Techniques (15).

The Pavon Corporation manufactures a binder which can be used with local aggregates to form a patching material. One 55 gal (208 l) drum, which costs about $75.00 can be combined with about 3.5 yd^3 (2.7 m^3) of aggregate. The resulting cost of the patching material is about $18.00/ton ($.02/kg) which is about $10.00/ton ($.011/kg) more than local patching materials. However, the use of this material may require fewer men and less equipment to place a satisfactory patch.

Public Technology, Inc., is evaluating a new thermoplastic-asphalt material for street patching in 27 jurisdictions across the country (5, 9). This material is a combination of asphalt and ethylene-vinyl acetate copolymers. An analysis of the cost-effectiveness of the new material indicates that, if the material has a road life twice that of current emergency patching materials, it is at least as cost effective as currently used materials.

Public Technology has also developed the framework which allows an agency to determine and analyze its current street patching costs and to evaluate the impact of a proposed change in labor, allocation, materials and equipment (5). This technique is useful for the evaluation of new patching materials and/or equipment.

The major disadvantages of these newer materials and techniques discussed above include the price of the material, specialized equipments for installation and the lack of familiarity by field maintenance crews. Maintenance crews usually resist the use of new materials and techniques. This is particularly true if handling and/or setting characteristics of the materials are different or if equipment clean up, repair and operation are different.

The cost of most of the binders of these new patching materials is high and normally results in a patching material that has an initial

cost 2 to 5 times that of conventional materials. However, materials are only one of the three major cost items for maintenance (equipment and labor are the other two). Increased cost of materials can be offset by reductions in equipment and labor costs. Additional benefit from an economic standpoint can be achieved if the life of the patch is increased by use of the new material.

12.3 Super-Penetrating Adhesives

This concept of localized repair involves the use of a sprayable material that is capable of penetrating cracked areas (i.e., alligator cracks, spalled areas, etc.) after which it sets-up and binds the cracks together. A wide range of possible materials exists for this highway application. Unfortunately, this type of material has not been evaluated on roadways.

Materials that appear suitable for this approach have been developed for and have been used for soil erosion, soil stabilization, rock binding and railroad ballasts. References 16 to 19 describe some of these materials and their application for agricultural purposes. References 20 and 21 illustrate applications for soil and rock binding for civil engineering applications while references 22 to 24 indicate potential applications in the rail ballast area.

Agricultural crops are vulnerable to damage by blowing soil immediately after seeding and during early growth. To solve and problem of soil erosion by wind in agricultural areas a number of chemical companies have developed products which when sprayed on the soil penetrate and bind the soil to prevent erosion (17, 18). Major manufacturers of products tested for this application include Witco Chemical, Union Carbide, Phillips Petroleum Company, Borden Chemical Company, Shell Chemical Company, Scott Paper Company, Armour Industrial Chemicals, Hercules, Inc., PPG Industries, Inc., Wica Chemicals, Douglas Oil Company, Ashland Chemical Company, DuPont, B. F. Goodrich Chemical Company, and Swift and Company. Other applications of these same soil stabilizing chemicals are in the areas of dust suppression of waste tailings at large mine operations, dust control around quarry operations, dust control in playgrounds, and dust control along

haul roads. Stabilization of sand dunes against wind erosion is also possible with some of these chemicals.

Limited laboratory tests have been conducted utilizing emulsion to stabilize ballasts. One of the emulsions tested is a solution of a thermoplastic butadiene-styrene block copolymer of relatively low solution viscosity and high tensile strength blended with a coumarone-indene resin (23). Tests have shown that these types of materials, when sprayed on stone ballast, will reduce permanent vertical settlement ties about 10 times over that of unstabilized ballast. Resistance to lateral displacement was 5 times greater for treated ballast as compared to untreated ballast (22).

Phillips Petroleum produces several emulsions for geotechnical uses which may have applications for pavement repair. These rubberized emulsion products and their basic use is described below:

1. Petroset SB - used for stabilizing soils from very fine particles through small gravel,

2. Petroset RB - used for stabilizing rock from small gravel through medium size and

3. Petroset AT - a rubberized asphalt concrete pavement after it has been spread and compacted.

Petroset SB can be purchased for $2.50 ($0.66 per ℓ) per gal in 55 (204 ℓ) gal drums or $2.25 ($0.59 per ℓ) per gal for truck load drum lots. These prices were effective December 1, 1976 and are F.O.B. Borger, Texas. Petroset AT is $3.50 ($0.92 per ℓ) per gal in tank car or tank truck lots. These prices are effective 31 March, 1977 and are F.O.B. Borger, Texas.

A detailed research and development program would be necessary to prove the applicability of this concept to highways. To establish the availability of these materials and their potential for higher applications numerous letters were prepared and contacts made. Several products identified in the above references are no longer manufactured by the chemical companies. No company indicated experience with an application as defined above.

If the concept of the super penetrating adhesive is utilized, it will in all likelihood be necessary to provide a wearing surface such as a chip seal or asphaltic concrete overlay.

12.4 Utility of Potential Rehabilitation Techniques

The utility of two localized repair methods were compared with a conventional patching operation. The new methods investigated were 1) use of super-penetrating adhesive plus a 1 in. (2.5 cm) overlay and 2) digout and repair with a new patching material plus a 1 in. (2.5 cm) overlay. Details of assumptions utilized to determine utilities follow.

12.4.1 Super-penetrating Adhesive

This localized repair method involved treatment of 10 percent of the pavement area with a super-penetrating adhesive plus a 1 in. (2.5 cm) overlay. The cost of materials that appear suitable for use as a super-penetrating adhesive is about 6 times that of an asphalt emulsion (21). Thus in-place costs for the material was assumed to be about 3 times that for a strip, fog seal (i.e. materials about 50 percent cost of the maintenance operation).

Maintenance costs and performance of this technique were assumed to be similar to a 2 in. (5 cm) conventional overlay while the initial SI value was selected to be that for a 1 in. (2.5 cm) conventional overlay. Energy required for material production was assumed to be that of a cutback asphalt plus that required for a 1 in. (2.5 cm) asphalt concrete overlay. Application energy was calculated to be that for a 1 in. (2.5 cm) overlay plus 2 times that required for a strip fog seal.

Some development costs were included but no additional equipment will be required for this technique. The repair time is assumed to be identical to a 1 in. (2.5 cm) overlay. The calculated utility for this method is 0.735.

12.4.2 Digout and Repair with a New Patching Material

This localized repair method involved the digout and repair of 10 percent of the pavement area plus a 1 in. (2.5 cm) overlay. The cost of the patching material in-place was assumed to be 2 times that of a conventional patching operation (5).

Maintenance costs and performance of this technique were assumed to be similar to a 3 to 4 in. (7.5 to 10 cm) conventional overlay while the initial SI value was selected to be that for a 1 in. (2.5 cm) conventional overlay. Energy requirements for materials production and the rehabilitation were assumed to be twice that of conventional patching materials.

Development costs were assumed to be one-half those required for the super-penetrating adhesive. No addiitonal equipment will be required for this technique. The repair time is assumed to be identical to a 4 in. (10 cm) AC overlay.

The calculated utility for this method is 0.722.

12.4.3 Digout and Repair with Conventional Patching Material

The localized repair method involved the digout and repair of 10 percent of the pavement area plus a 1 in. (2.5 cm) overlay. The cost of the patching was determined based on a 6 in. (15 cm) thick patch made with asphaltic concrete.

Maintenance costs and performance of this technique were assumed to be similar to a 3 to 4 in. (7.5 to 10 cm) conventional overlay while the initial SI value was selected to be that for a 1 in. (2.5 cm) conventional overlay. The life of a pavement treated with this technique was slightly less than that for digout and repair with new patching material. Energy requirements were based on those required for asphalt concrete.

Since this is a conventional method no development or equipment costs are required. The repair time is assumed to be identical to a 4 in. (10 cm) overlay.

The calculated utility for this method is 0.778.

12.4.4 Conventional Asphaltic Concrete Overlays

Conventional asphaltic concrete overlays for pavements with similar distress and identical traffic volumes have the following utilities (see chapter 4).

1 in. (2.5 cm) asphaltic concrete overlay - 0.705

2 in. (5 cm) asphaltic concrete overlay - 0.737

12.4.5 Comparison of Utility Values

A comparison of the different utility values is given in Table 12-2. Based on the stated assumptions neither of the two new techniques appear to be as good as the conventional techniques of digout and replace or just simply a 2 in. (5 cm) AC overlay. Of course this conclusion is only as valid as the input assumptions and data. Should they change the results could change.

12-2. Utility Values For Techniques to Solve Specific Localized Repair Problems

Technique	Utility Values for Localized Repair of Flexible Pavement
A. Potential Techniques	
Super Penetrating adhesive plus 1 in. (2.5 cm) AC Overlay	0.735
Digout and Repair with New Patching Material plus 1 in. (2.5 cm) AC Overlay	0.722
B. Current Techniques	
Digout and Repair with Conventional Patching Material plus 1 in. (2.5 cm) AC Overlay	0.778
1 in. (2.5 cm) AC Overlay	0.705
2 in. (5 cm) AC Overlay	0.737

It is suggested that additional research in new patching materials be delayed until performance data and appropriate economic analyses are made on the patching materials developed by Public Technology (5, 9). Furthermore, it is suggested that a formalized information exchange program be initiated among states, counties and cities such as that presently utilized AASHTO for new product evaluation.

The relatively high utility value for the super-penetrating adhesive indicates promise. Information gained from the use of chemicals for agricultural purposes and for dust control will be helpful. An initial project on the order of $100,000 to $200,000 could provide sufficient information to more confidently assess the potential of this technique.

13. SENSITIVITY ANALYSIS OF THE DECISION ANALYSIS PROGRAM

13.1 Introduction

Computer program UDAREM was composed to model an actual decision making process which ranks rehabilitation techniques according to their relative utility. A sensitivity analysis of the program was undertaken to determine whether or not the computerized model is a reasonable approximation of the actual decision making process.

The two major questions that must be answered by this sensitivity analysis are:

1. What effect does a change of weighting factors have on the expected level of utility and on the ranking of specific techniques?
2. What effect does a change of the input decision variables have on the expected value of the utility?

The first question was answered by a comparison of the utilities derived from weighting factors selected by project staff and the utilities determined using the assumed weighting factors from XDOT, the hypothetical state department of transportation (see table 2-17). Answers to the first question will show how likely it is that different recommendations or rankings would have been made by transportation agencies other than TTI, if they had been making the decisions.

The second question was answered by a study of two promising rehabilitation techniques which had low utility values, one for flexible and the other for rigid pavements. The flexible pavement technique is microwave heating and the rigid pavement technique is prestressing. The answers to this question showed how the utilities and rankings might change if (presumably) more accurate information were obtained on the decision variables (such as cost, performance, energy, safety, and so on).

13.2 Probable Range of Utilities

As discussed in chapter 2, the uncertainty concerning specific values assigned to each of the 17 decision criteria, was handled by assuming all the variables to have a Beta probability density function. The result of this probabilistic approach was the generation of mean utility values for each technique (termed the technique's "expected utility, $\bar{u}$"), and a

variance for each technique-which indicated the relative range of values that the utility could have. Most of the utility variances were quite small (0.0010 or less). The standard deviations were, for the most part less than 0.04 which means there is a 68% probability that the actual utility value lies in the range of $\bar{u} \pm 0.04$ and a 95% probability that the actual value lies in the range of $\bar{u} \pm 0.08$. Thus for an expected utility of 0.700, there is a 68% probability that the actual utility value lies between 0.660 and 0.740 and a 95% probability that the actual utility value lies between 0.620 and 0.780. As illustrated by this example, the range of uncertainty in the overall utility values was quite small, which meant that different techniques could be compared on the basis of their expected utilities without considering the variances.

13.3 Sensitivity to Weighting Factors

In chapter 2, two sets of weighting factors were presented, a hypothetical state transportation agency's (termed XDOT) and the research team's (see Table 2-17). To ascertain the difference in utility values from the two sets of weighting factors, all techniques were evaluated using both sets of weighting factors. The following three studies were made to determine the sensitivity of the utilities and relative ranking to this difference in weighting factors.

1. An examination of whether rehabilitation techniques listed as "Potential" with the research team's weighting factors would not be so highly regarded with XDOT's weighting factors.
2. A series of tests for statistically significant differences between utilities calculated using the two sets of weighting factors.
3. A study of the change of ranking of each rehabilitation technique within the list of techniques considered in each chapter of this report.

13.3.1. Potential Technique Decisions. In only one case was the decision concerning the potential technique reversed. Table 13-1 shows the comparison of three techniques. The decision based on the research team weighting factors was that there is not a significant difference in the utility of the three techniques. However, considering

Table 13-1. Utility Values for Different Weighting Factors

Technique	Research Team Weighting Factor	XDOT Weighting Factors
Chapter 3-Potential Techniques		
Electro-Osmosis	0.706	0.656
Electro-Chemical	0.702	0.707
Chapter 3-Current Techniques		
Dig out and Replace	0.706	0.752

the utility values based on the XDOT weighting factors the decision would be that the existing technique is more feasible.

13.3.2. Test for Significant Differences in Utilities. There were several significant differences noted in the utilities calculated with the two sets of weighting factors. A significant difference was determined by performing a statistical analysis, by chapters, of the utility values using a paired student's t-test. In each chapter, several lists of rehabilitation techniques were presented -- some for segmental and others for localized applications. Most chapters examined a variety of typical locations; urban, suburban and rural. Each of these variations was tested for significant differences and the results are recorded in Table 13-2.

The mean difference in utility values ($\bar{d}$) was determined by subtracting the XDOT utility value (U_X) from the research team utility value (U_{RT}) for each technique in a given list; then summing the differences algebraically and finally dividing by the number of techniques in the list (N). The standard deviations (SD) were determined by using the standard small data statistical equations.

The following assumptions were made for this analysis:

a. The utilities are normally distributed

b. Ho (null hypothesis): $U_{RT} = U_X$

c. H_1 (alternative hypothesis): $U_{RT} > U_X$ or $U_X > U_{RT}$

d. α (level of significance): 0.05

The student's t-statistic was calculated from the equation:

$$t_{calc} = \frac{\bar{d} - 0}{SD/\sqrt{N}} \qquad (13.1)$$

where each of the variables have been defined in the preceding discussion. These calculated values were compared with the standard t-table values for a two tailed test. If the calculated value was larger than the tabulated value, the difference was considered to be probably significant.

The t-test is made assuming that the two sets of data that are compared are normally distributed with equal variances. Tests for normality on the segmental utilities showed that all were normally distributed and this means that the conclusions to be drawn below are statistically

Table 13-2. Summary of Statistical Tests for Significant Differences Between Utilities.

Chapter	Highway Level[a]	Rehabilitation Type[b]	Mean Difference in Utilities $\bar{d}$	Standard Deviation SD	N	Significant Difference
3	U	S	0.040	0.037	15	Yes
3	U	L	0.014	0.036	6	No
4	U	S	0.051	0.013	2	No
4	Su	S	0.029	0.006	2	No
4	Ru	S	0.020	0.031	18	Yes
5	Su	S	0.021	0.045	2	No
6	Su	S	-0.006	0.017	7	No
6	Su	L	-0.013	0.002	3	Yes
7	Su	S	0.021	0.045	3	No
8	U	S	-0.011	0.029	5	No
8	Ru	S	-0.037	0.036	5	No
9	Ru	S	0.025	0.001	2	Yes
10	U	S	-0.002	0.018	4	No
10	Ru	S	0.006	0.028	5	No
11	U	S	0.026	0.018	4	No
11	R	S	-0.007	0.011	4	No
12	Su	S	0.025	0.008	3	Yes

[a] U = Urban, Su = Suburban, Ru = Rural

[b] S = Segmental, L = Localized

sound.

Statistically significant differences occurred in chapters 3, 4, 6, 9, and 12 under one or more of the assumed conditions. Furthermore the XDOT utilities were larger than the research team utilities in chapters 6, 8, and 10.

Thus, despite the fact that only small differences are observed between the XDOT utilities and those of the research team, in many cases these differences are significant, indicating that the utility values are sensitive to the set of weighting factors adopted.

13.3.3. Change of Rank. A further indication of the sensitivity of the program to a change of weighting factors is how the change affects the position of a technique in a list that is ranked in order of decreasing utility. It is possible that differences in utility from one set of weights to another may remain small but the relative preference of one technique over another may change significantly. The change in ranking of techniques was examined by ranking each list of utilities separately and subtracting the research team ranking from the XDOT ranking. This number shows by how much the rank of each technique must change in order to be identical with the corresponding list. The sum of the absolute value of these changes (C) is an indication of how much reordering must be done to change the ranking of one list of techniques to be the same as the other. The average change of rank ($\overline{C}$) is obtained by dividing the sum of the absolute values by the number of techniques in the list (N).

The length of the list plays a large part in determining how many positions a given technique must change on the average. The longer the list, the larger will be the average change of position. In order that comparisons of relative position change might be made, the average position change ($\overline{C}$) was divided by the number of techniques in a list (N) to get a change ratio (CR). The results of all of these calculations is given in Table 13-3.

Significant changes of ranking occurred in all lists that numbered more than about 5. This probably constitutes a lower limit of the validity of this type of analysis, since with a list of 4 or less, the choices of best, second best, and so on are probably much more clear.

Table 13-3. Summary of Change of Rank With Different Sets of Weighting Factors

Chapter	Highway Level[a]	Rehabilitation Type[b]	C[c]	$\bar{C}$[d]	CR[e]	N[f]
3	U	S	14	0.93	0.062	15
3	U	L	2	0.33	0.056	6
4	U	S	0	0	0	2
4	Su	S	0	0	0	2
4	Ru	S	36	2.00	0.111	18
5	Su	S	0	0	0	2
6	Su	S	2	0.29	0.041	7
6	Su	L	0	0	0	3
7	Su	S	0	0	0	3
8	U	S	0	0	0	5
8	Ru	S	4	0.80	0.160	5
9	Ru	S	0	0	0	2
10	U	S	2	0.50	0.125	4
10	Ru	S	4	0.80	0.160	5
11	U	S	0	0	0	4
11	Ru	S	0	0	0	4
12	Su	S	0	0	0	3

[a] U = Urban, Su = Suburban, Ru = Rural

[b] S = Segmental, L = Localized

[c] C = Sum of absolute value of changes of rank

[d] $\bar{C}$ = Average change of rank

[e] CR = $\bar{c}/N$

[f] N = Number of techniques in a list

The average change of rank ($\overline{C}$) in the rest of the lists varied from 0.29 to 2.00 and the change ratio (CR) varied between 0.04 and 0.16. An average change of rank of 2.0 means that on the average each technique is altered two positions, up or down, in a list as a result of a change of weighting factors. Thus, a technique ranked as number 3 in one list may be ranked as number 1 or number 5 on the other.

Because a decision must be normally made to use one technique to the exclusion of all others on a given project, an average rank change of 2.0 can make a significant difference upon what is decided. Thus, it may be concluded that the choice of weighting factors can have a significant effect upon the utilities calculated by the decision analysis program. This is as it should be, since in properly modeling a decision making process, the program should be sensitive to the weights that decision makers apply to different decision criteria, and in particular, should result in different rankings of alternatives.

13.4 Sensitivity to Decision Criteria

The next important question to be answered is concerned with the sensitivity of the calculated utilities to changes in individual decision criteria. In answering the question it is as important to indicate which of these criteria have most influence upon the overall utility as it is to determine the actual size of the change.

In order to have a consistent basis upon which to judge the relative contribution of each decision criterion, the following rules were adopted:

1. on those decision criteria with a continuous scale, determine the change of utility when the original values are halved and doubled. These criteria include the costs, times, and energies.

2. on those decision criteria with a subjective scale from 1 to 7, determine the change of utility due to an upgrading or downgrading of the rating by one integer rating. These criteria include reworkability, safety, noise and pollution.

3. on user benefits and energy criteria, determine the change of utility due to a change of traffic by factors of 1/2 and 2.

4. on the level of pavement service criterion, determine the change of utility due to a change in acceptable life by factors of 1/2 or 2.

5. Material energy will not vary with a given technique since both the energy equivalent and application rate are specified with a given technique. Consequently, there will be little variability of this decision criterion with a given technique and thus there is little practical sensitivity.

The original value of utility for Prestress Pavement was 0.545 (See Chapter 9) when applied in a rural traffic situation. The original value of utility for Microwave Heating was 0.654 (See Chapter 4). As neither of these two techniques were found promising in this study they were chosen to see what effect significantly changing input data would have on the resulting utility of each technique. All other calculated utilities should be compared with the datum values obtained in the project. The results of the calculations are shown in Tables 13-4 and 13-5.

It will be observed in both tables that all decision criteria are negatively correlated with utility except user benefits, user energy and pavement life. Negative correlation means that an increase in a value of a decision criterion causes a decrease of utility. User benefits and energy are positively correlated because there are greater amounts of benefits and energy savings at higher traffic levels. Pavement life is also positively correlated for obvious reasons.

The percentages of changes of utility ranged from 0 to 6.3 percent (see Tables 13-4 and 13-5), a rather small percentage for a change of decision criterion by a factor of 2. However, it will be recalled from the previous section of this chapter that a fairly small change in utility can cause a significant change in the ranking of a rehabilitation technique. Therefore, it is useful to review the calculated results in Tables 13-4 and 13-5 to determine which decision criteria can cause the largest changes in utility.

In both the flexible and rigid pavement techniques, the most sensitive decision criteria were:

1. Maintenance Cost (No. 4)
2. Level of Pavement Service (No. 8)
3. Safety During Construction (No. 15)
4. Rehabilitation Energy (No. 12)

Table 13-4. Sensitivity of Prestressed Pavement Utilities (Ch 9) to Variations In Decision Criteria

No.	Decision Criterion	Utilities[a]		Utility Change	
		Low Level	High Level	Low Level	High Level
1	Development Cost	0.550	0.536	+0.005	-0.006
2	Capital Equipment Cost	0.545	0.545	0.000	0.000
3	Construction Cost	0.550	0.545	+0.005	0.000
4	Maintenance Cost	0.580	0.515	+0.035	-0.030
5	User Benefits	0.540	0.551	-0.013	-0.002
6	Time of Development	0.546	0.545	+0.001	0.000
7	Life	0.531	0.545	-0.014	0.000
8	Level of Pavement Service	0.566	0.512	+0.021	-0.033
9	Traffic Density-Const. Time	0.546	0.544	+0.001	-0.001
10	Reworkability	0.563	0.533	+0.018	-0.012
11	User Energy Savings	0.540	0.551	-0.013	-0.002
12	Rehabilitation Energy	0.560	0.524	+0.015	-0.021
13	Material Energy	--	--	--	--
14	Safety Improvement	0.553	0.532	+0.008	-0.013
15	Safety During Construction	0.563	0.527	+0.018	-0.018
16	Noise	0.552	0.538	+0.007	-0.007
17	Pollution	0.552	0.535	+0.007	-0.010

[a]The original value of utility is 0.545 and is based on rural weights.

Table 13-5. Sensitivity of Microwave Heating Utilities (Ch 4) to Variations in Decision Criteria

No.	Decision Criterion	Utilities[a]		Utility Change	
		Low Level	High Level	Low Level	High Level
1	Development Cost	0.657	0.647	+0.003	-0.007
2	Capital Equipment Cost	0.659	0.652	+0.005	-0.003
3	Construction Cost	0.654	0.635	+0.000	-0.019
4	Maintenance Cost	0.686	0.618	+0.032	-0.036
5	User Benefits	0.649	0.659	-0.005	+0.005
6	Time of Development	0.664	0.652	+0.010	-0.002
7	Life	0.643	0.654	-0.011	0.000
8	Level of Pavement Service	0.667	0.639	+0.013	-0.015
9	Traffic Density-Constr. Time	0.654	0.654	0.000	0.000
10	Reworkability	0.657	0.642	+0.003	-0.012
11	User Energy Savings	0.649	0.659	-0.005	+0.005
12	Rehabilitation Energy	0.671	0.635	+0.017	-0.019
13	Material Energy	--	--	--	--
14	Safety Improvement	0.657	0.641	+0.003	-0.013
15	Safety During Construction	0.669	0.634	+0.015	-0.020
16	Noise	0.659	0.646	+0.005	-0.008
17	Pollution	0.662	0.641	+0.008	-0.013

[a]The original value of utility is 0.654 and is based on suburban weights.

The next most sensitive criteria are different depending upon whether rigid or flexible pavement techniques are examined. With prestressed pavement, the safety, noise, and pollution decision criteria are important whereas with microwave heating, pollution and construction costs are next in importance.

A certain amount of insensitivity can be observed which are due to peculiarities of these two techniques. For example, the capital equipment cost of prestressed pavement is insensitive because no such cost was determined to be necessary for this technique which has already been developed. Another example is the traffic density - construction time requirement which was insensitive because of the low traffic volume and the relatively short estimated construction times. These decision criteria never got into the sensitive range of the utility curves. A similar observation could be made of the short time of development required for these two techniques. The utility of the prestressed pavement technique became insensitive to an increase of construction cost because that cost was nearly off the end of the utility curve where utility drops to zero.

This latter insensitivity near the ends of the utility curve is a limitation of the way utilities were calculated. The difficulty just noted could be eliminated by allowing the utility curve to become negative as was done with the impact decision criteria.

13.5 Summary

The decision analysis program is sensitive to variations in weighting factors and in values of decision criteria. Although the overall utilities may not change much, changes in weights and decision criteria can cause a reordering of priorities, which is where the real sensitivity of the computer program becomes most evident. On the basis of the sensitivity analysis presented in this chapter, a considerable amount of reordering may be appropriate if more accurate information is obtained on maintenance costs, pavement service and distress, safety during construction, and rehabilitation energy.

14. CONCLUSIONS AND RECOMMENDATIONS

14.1 Conclusions

During the course of this investigation some 92 potential techniques for rehabilitating pavements without overlays were identified, analyzed, and their value assessed. In order to handle the myriad of ideas and pavement conditions a systems analysis was developed. Following are the major conclusions reached in this project. They are based on the particular conditions enumerated in the report. Generalizations beyond the stated conditions may not be warranted.

1. In order to make rational rehabilitation decisions in today's complex world, the decision maker should consider 17 different factors (termed decision criteria) which may be grouped in four categories (termed attributes). They are: Costs (which consist of (1) development cost, (2) capital equipment cost, (3) construction cost, (4) maintenance cost, and (5) user savings); Performance (which consists of (6) time of development, (7) life, (8) level of pavement service, (9) traffic density-construction time, and (10) expected reworkability); Energy (which consists of (11) user energy savings, (12) rehabilitation energy, and (13) material energy); and Impact (which consists of (14) safety improvements, (15) safety during construction, (16) noise, and (17) pollution).

2. Using a hierarchical weighting method of utility systems analysis, a computer program was developed which considers all 17 (or any other number) decision criteria for each technique and outputs on overall mean utility value and variance under conditions of uncertainty as to the input data. The computer program is fast, inexpensive, extremely flexible, and easy to use.

3. The computer program was designed to be insensitive to the uncertainty of the specific values of each input data, yet sensitve to the change in data values between techniques. Thus even though a decision maker may not know exactly how much a particular technique might cost, through estimates of optimistic, most probable, and pessimistic values for cost (or any other criterion) its overall utility will exhibit only a small variance.

4. The decision analysis program is sensitive to variations in weighting factors and in values of decision criteria. Although the overall

utilities may not change much, changes in weights and decision criteria can cause a re-ordering of priorities, which is where the real sensitivity of the computer program becomes most evident.

5. Of the 92 potential techniques examined, 30 of them were discarded in the preliminary screening process, and 62 were analyzed in detail for their potential value. A number of potential techniques were found to be sufficiently similar to one another to group together. After grouping these, the number of distinct techniques to be investigated totaled 39.

6. A total of 19 promising potential techniques were found to rehabilitate pavements experiencing specific types of distress under certain conditions. A summary of these potential techniques and applicable conditions is given in Table 14-1. Each of these techniques exhibited utility values equal to or higher than the utility values of currently used techniques.

7. Under certain conditions, currently used techniques offered the greatest potential for solving specific problems. These techniques and applicable conditions are summarized in Table 14-2.

8. The following 5 potential techniques, although found to be as good as or better than current techniques, were not found to be as good as other potential techniques (see Table 14-1) to solve the same type of distress:

a. Heater planing of flexible pavements
b. Cold milling of flexible pavements
c. Stress relieving interface
d. Heavy rolling of flexible pavements
e. Crack sealing of flexible pavements

9. At this time the following 15 techniques did not offer any potential for rehabilitating pavements:

a. High strength, post-hole piles
b. Heat treatment of plastic subgrades
c. Microwave sealing of flexible pavements
d. Ultra-violet irradiation of flexible pavements
e. Cold planing of flexible pavements
f. Hot milling of flexible pavements
g. Reverse pavement cross slope
h. Polymer impregnated concrete surface
i. Polymer modified concrete surface

Table 14-1. Summary of Promising Techniques and Applicable Conditions

Potential Technique	Rehabilitation Type [a]	Highway Type [b]	Highway Level [c]	Applicable Distress Conditions
High Pressure Portland Cement Grouting	S	R	U	1. Joint or edge faulting or pumping 2. Porus subbase too weak
Penetration Chemical Grouting	S	R,F	U	1. Edge pumping (R) 2. Porous subbase too weak (F)
Electro-Osmosis plus 1 in. (2.5 cm) AC Overlay	L	F	U	Highly plastic clay subgrade too weak
Electro-Chemical plus 1 in. (2.5 cm) AC Overlay	L	F	U	Highly plastic clay subgrade too weak
Rejuvenating Agents	S,L	F	Su	Surface loss and deterioration
Reworked Surface	S,L	F	Su	1. Distortion of profile or cross-section 2. Surface cracking 3. Asphalt bleeding 4. Holes and indentions
Special Slurry seals	S,L	F	Su	1. Surface loss and deterioration 2. Surface cracking

[a] S = Segmental, L = Localized

[b] R = Rigid Pavement, F = Flexible Pavement

[c] U = Urban, Su = Suburban, Ru = Rural

Table 14-1. Summary of Promising Techniques and Applicable Conditions (Continued)

Potential Technique	Rehabilitation Type [a]	Highway Type [b]	Highway Level [c]	Applicable Distress Conditions
Replace Wheel Path	S,L	F	Su	1. Distortion of road profile or cross-section 2. Asphalt bleeding 3. Holes and indentions
Change Location of Lane Markings	S	F	Su	1. Rutting (minor) 2. Edge fatique damage
Joint Restoration with Improved Joint Sealants	S,L	R	Su	Joint raveling, spalling, and pumping
Horizontally-Bored Sleeper Slab and Joint Restoration	S	R	Su	Joint raveling, spalling, pumping, and minor faulting
Precast Joint Assembly	L	R	Su	1. Joint or crack blowups, spalling, and pumping 2. Compression cracks
Improved Load Transfer Devices	S	R	Su	Joint raveling, spalling, and pumping
Full Depth Precast Elements (Plain Joints)	L	R	U,Su,Ru	1. Blowups, compression failure 2. Faulting
Full Depth Precast Elements (Tied Joints)	L	R	U,Su,Ru	1. Blowups, compression failure 2. Faulting

[a] S = Segmental, L = Localized

[b] R = Rigid Pavement, F = Flexible Pavement

[c] U = Urban, Su = Suburban, Ru = Rural

Table 14-1. Summary of Promising Techniques and Applicable Conditions (Continued)

Potential Technique	Rehabilitation Type [a]	Highway Type [b]	Highway Level [c]	Applicable Distress Conditions
Partial Depth Precasts Slabs	L	R	U,Su,Ru	Spalling at transverse joints
Rout Out and Fill Cracks in Jointed Pavement	S	R	U,Su,Ru	Cracking in jointed pavement
Pressure Injection of Epoxy in CRCP	S	R	U,Su,Ru	Wide Cracks in CRCP
Pressure Injection of Epoxy Followed by Sawing New Joint in Jointed Pavement	S	R	U,Su	Transverse cracking in jointed pavement

[a] S = Segmental, L = Localized

[b] R = Rigid Pavement, F = Flexible Pavement

[c] U = Urban, Su = Suburban, Ru = Rural

Table 14-2. Summary of Presently Used Techniques to Solve Certain Problems

Current Technique	Rehabilitation Type [a]	Highway Type [b]	Highway Level [c]	Applicable Distress Condition
2 in. (5 cm) AC Overlay	S,L	F,R	U	1. Granular, porous subgrade too weak (S) 2. Highly plastic lay subgrade too weak (S) 3. Localized sublayer failure (L,R)
Level/Roughen Surface	S,L	R	Su	Loss of PCC pavement surface
Partial-Depth, Cast-In-Place Concrete Patch	L	R	Su	Spalling at transverse joints
6 in. (15 cm) AC Overlay	L	R	Ru	Preemptive repair of PCC pavement before general failure
Tied Concrete Shoulders	S	R	U,Ru	Pavement edge pumping, low fatique life
Dig Out and Repair with Conventional Patching Plus 1 in. (2.5 cm) AC Overlay	L	F	Su	Localized failure in flexible pavement

[a] S = Segmental, L = Localized

[b] R = Rigid pavement, F = Flexible pavement

[c] U = Urban, Su = Suburban, Ru = Rural

j. Fiber reinforced concrete surface

k. Prestressing existing pavements

l. Penetration grouting beneath cracks in CRCP

m. Edge beam techniques

n. Super penetrating adhesive plus AC overlay

o. Dig out and repair with new patching material plus AC overlay

14.2 Recommendations

14.2.1. Development and Implementation. Based on the conclusions reached in this investigation the following recommendations were made:

1. The systems analysis utilizing utility theory, developed on this project, should be refined and further optimized by developing utility input values and weighting factors which meet the needs and goals of specific decision makers, e.g. specific state transportation agencies.

2. The 19 promising potential techniques for rehabilitating pavements should be evaluated in the field. They should be utilized under the applicable conditions stated. Detailed records should be kept on the cost, performance, energy, and impact of each technique. These data should be used to reevaluate the utility of each technique.

3. Of the 19 promising techniques, the following five should be given first priority. They each offer the greatest potential for solving one or more of our most troubling distress manifestations.

a. Rejuvenating agents for flexible pavements

b. Horizontally-bored sleeper slab and joint restoration for rigid pavements

c. Precast joint assemblies for rigid pavements

d. Change the location of lane markings

e. Reworked surface of flexible pavements

14.2.2 Further Research

1. Although this project did not indicate the following techniques to have sufficient potential to warrant field demonstration, further research into each technique is recommended. If technological breakthroughs could be achieved they could solve some of our more troubling types of distress.

a. Prestressing existing rigid pavements

b. Use of super penetrating adhesives for flexible pavements

c. Microwave sealing of flexible pavements

2. The use of utility theory should be expanded to assist decision makers involved in all aspects of transportation engineering, including:

a. Pavement structure design strategies
b. Bridge design strategies
c. Geometric design strategies
d. Highway and bridge maintenance strategies
e. Safety improvement strategies
f. Multi-modal warrants and needs
g. Transportation budget explanations
h. Transportation budget allocations
i. Transportation economic modelling
j. Traffic planning

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FEDERALLY COORDINATED PROGRAM OF HIGHWAY RESEARCH AND DEVELOPMENT (FCP)

The Offices of Research and Development of the Federal Highway Administration are responsible for a broad program of research with resources including its own staff, contract programs, and a Federal-Aid program which is conducted by or through the State highway departments and which also finances the National Cooperative Highway Research Program managed by the Transportation Research Board. The Federally Coordinated Program of Highway Research and Development (FCP) is a carefully selected group of projects aimed at urgent, national problems, which concentrates these resources on these problems to obtain timely solutions. Virtually all of the available funds and staff resources are a part of the FCP, together with as much of the Federal-aid research funds of the States and the NCHRP resources as the States agree to devote to these projects.*

FCP Category Descriptions

1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems connected with the responsibilities of the Federal Highway Administration under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

2. Reduction of Traffic Congestion and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by keeping the demand-capacity relationship in better balance through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

3. Environmental Considerations in Highway Design, Location, Construction, and Operation

Environmental R&D is directed toward identifying and evaluating highway elements which affect the quality of the human environment. The ultimate goals are reduction of adverse highway and traffic impacts, and protection and enhancement of the environment.

4. Improved Materials Utilization and Durability

Materials R&D is concerned with expanding the knowledge of materials properties and technology to fully utilize available naturally occurring materials, to develop extender or substitute materials for materials in short supply, and to devise procedures for converting industrial and other wastes into useful highway products. These activities are all directed toward the common goals of lowering the cost of highway construction and extending the period of maintenance-free operation.

5. Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

Structural R&D is concerned with furthering the latest technological advances in structural designs, fabrication processes, and construction techniques, to provide safe, efficient highways at reasonable cost.

6. Prototype Development and Implementation of Research

This category is concerned with developing and transferring research and technology into practice, or, as it has been commonly identified, "technology transfer."

7. Improved Technology for Highway Maintenance

Maintenance R&D objectives include the development and application of new technology to improve management, to augment the utilization of resources, and to increase operational efficiency and safety in the maintenance of highway facilities.

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